

A

BENEFICIAL USE OF RECYCLED MATERIALS
IN CONCRETE MIXTURES

by

Patrick Lance Maier

B.S. Civil Engineering, University of Colorado Denver, 2008

A thesis submitted to the
University of Colorado Denver
In partial fulfillment
of the requirements for the degree of
Master of Science, Structural Engineering
Civil Engineering

2011

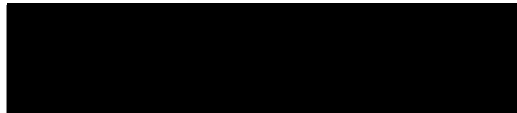
This thesis for the Master of Science

degree by

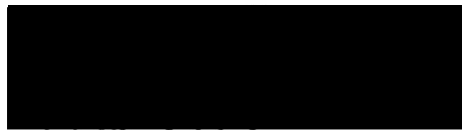
Patrick L. Maier

has been approved

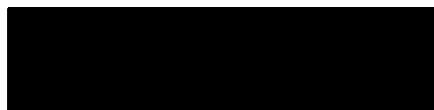
by



Dr. Stephan A. Durham



Dr. Kevin L. Rens



Dr. Chengyu Li

Date

Maier, Patrick Lance (M.S. Structural, Civil Engineering Department)

Beneficial Use of Recycled Materials in Concrete Mixtures

Thesis directed by Dr. Stephan A. Durham

ABSTRACT

The need to produce concrete mixtures with recycled materials is becoming more important than ever before. Not only does using recycled materials in concrete mixtures create landfill avoidance, but it decreases the depletion of virgin raw materials. The basis for this research was to investigate the effects of using recycled materials, in varying amounts, on the fresh and hardened concrete properties. This research includes the design of concrete mixtures composed of varying amounts of recycled material replacements. The recycled materials in this study consisted of ground granulated blast furnace slag (GGBFS), recycled concrete (crushed hardened concrete) and crushed waste glass. The GGBFS was used as a replacement for the cement. The recycled concrete and waste glass were used to replace the coarse and fine aggregates, respectively. The concrete mixtures designed ranged from a twenty five percent replacement to one hundred percent replacement with recycled materials. These mixtures were compared against a standard concrete mixture using cement and virgin aggregates. For comparison purposes, all mixtures were held constant in regards to water to cementitious ratio. The fresh concrete properties examined included slump, air content and unit weight. The hardened properties examined included compressive strength, rate of strength gain, freeze-thaw durability, permeability, and alkali-silica reactivity potential.

A concrete mixture composed entirely of recycled materials was developed. This concrete mixture developed substantial strength and durability and is comparable to a normal strength concrete mixture in several aspects. This concrete made from

100% recycled materials was a very low permeable concrete with a compressive strength of 4300 psi (29.6 MPa). A concrete composed of 50% and 75% recycled materials that achieved strengths of nearly 7000 psi (48 MPa) and 6300 psi (43.4 MPa) respectively were also developed. The beneficial and negative effects of using recycled aggregates and GGBFS in a concrete mixture were determined. The deleterious expansions caused by the waste glass reacting with alkalis in the cementitious paste (ASR) were also determined. It was found that GGBFS, when used at replacement levels of 50%, eliminated these concerns when waste glass is used at even 100% aggregate replacement levels.

The point at which replacement with recycled materials becomes detrimental to the concrete mixture, in regards to strength, durability and workability was determined. A replacement of 50% recycled materials was determined to be an optimum replacement amount for concrete. The use of recycled materials was determined to be a benefit with regards to strength and durability up to 50% when compared with a normal concrete made from virgin materials. However, it was shown that even a concrete with recycled materials in excess of 50% can be very beneficial and comparable to a normal, regular strength concrete. Although freeze-thaw durability's decreased for concretes made with recycled contents in excess of 75%, the permeability's of these mixtures are extremely low and when coupled with substantial strength, these concretes would be suitable for use in many applications.

This abstract accurately represents the content of the candidate's thesis. I recommend its publication.

Signed

A black rectangular box redacting the signature of Dr. Stephan A. Durham.

Dr. Stephan A. Durham

DEDICATION PAGE

I dedicate this thesis to my family and friends for their never ending support. To my mother for her continued support and showing me that giving up is not an option. To my brothers for their inspiration to become an engineer. To the many relationships that have been severed and the friendships that were lost over the many years during my undergraduate and graduate studies.

ACKNOWLEDGMENT

I would like to thank my academic advisor, Dr. Stephan A. Durham. His continued support throughout the years has encouraged me and many others. I would like to thank my long time undergraduate advisor Dr. Kevin Rens for his patience, support and humor over the last decade. I would like to acknowledge Dr. Jonathan Wu, for his inspiration and always having his door open for a wondering mind. I would like to thank Dr. Li for his support and participating on my thesis committee. I would like to thank Dr. N.Y. Chang for his guidance, support and kindness through the years. I would also like to thank Rui Liu, Brian Volmer, Driss Majdoub and Adam Kardos for their support and much appreciated help during my research

I would like to thank Katie Bartojay and Bill Kepler for their support during this research. Without their experience, knowledge and contacts in the concrete industry, this research would not have been successful. I would like to thank Morgan Johnson and LEHIGH Cement for their generous contribution of ground granulated blast furnace slag for this research. I would like to thank Tony Able and Rocky Mountain Bottle Recycling for their contributions of waste glass. I would also like to thank John Kent and Oxford Recycling for their donations of recycled concrete. I would like to especially thank Bud Werner and his remarkable staff at CTL Thompson for their generous help testing for potential ASR.

Additionally, I would like to thank the faculty and staff of the University of Colorado Denver, Civil Engineering Department for their support and guidance throughout my educational career at UCD.

TABLE OF CONTENTS

Figures.....	xi
Tables.....	xiii

Chapter

1.	Introduction.....	1
2.	Literature Review.....	6
2.1	Preface.....	6
2.2	Ground Granulated Blast Furnace Slag (GGBFS)	6
2.2.1	Production.....	6
2.2.2	Physical, Chemical and Reactive Properties.....	7
2.2.3	The Effects of GGBFS on Fresh Concrete Properties.....	10
2.2.3.1	Slump.....	10
2.2.3.2	Air Content.....	11
2.2.3.3	Time of Set.....	12
2.2.3.4	Temperature.....	13
2.2.4	The Effects of GGBFS on Hardened Concrete Properties.....	14
2.2.4.1	Strength.....	14
2.2.4.2	Permeability.....	21
2.2.4.3	Freeze-Thaw Durability.....	24
2.2.4.4	Resistance to Sulfate Attack.....	25
2.2.4.5	Alkali-Silica Reactivity.....	29
2.2.5	Summary.....	33
2.3	Waste Glass as Aggregate.....	34

2.3.1	Production.....	35
2.3.2	Physical and Chemical Properties.....	36
2.3.3	The Effects of Waste Glass on Fresh Concrete Properties.....	38
2.3.3.1	Slump.....	38
2.3.3.2	Air Content.....	40
2.3.4	The Effects of Waste Glass on Hardened Concrete Properties.....	41
2.3.4.1	Strength.....	41
2.3.4.2	Permeability.....	44
2.3.4.3	Freeze-Thaw Durability.....	46
2.3.4.4	Alkali-Silica Reactivity (ASR).....	47
2.3.5	Summary.....	50
2.4	Recycled Concrete as Aggregate (RCA).....	52
2.4.1	Production.....	52
2.4.2	Physical and Chemical Properties.....	53
2.4.3	The Effects of RCA on Fresh Concrete Properties.....	54
2.4.3.1	Slump.....	54
2.4.3.2	Air Content.....	57
2.4.4	The Effects of RCA on Hardened Concrete Properties.....	58
2.4.4.1	Strength.....	58
2.4.4.2	Permeability.....	67
2.4.4.3	Freeze-Thaw Durability.....	71
2.4.4.4	Alkali-Silica Reactivity (ASR).....	74
2.4.5	Summary.....	74
2.5	Expectations.....	76
3.	Problem Statement.....	78
3.1	Statement.....	78

4.	Experimental Plan.....	80
4.1	Design Summary.....	80
4.2	Material Properties.....	81
4.2.1	Ground Granulated Blast Furnace Slag (GGBFS).....	81
4.2.2	Portland Cement.....	82
4.2.3	(Virgin) Coarse and Fine Aggregates.....	84
4.2.4	(Recycled) Coarse and Fine Aggregates.....	84
4.2.4.1	Waste Glass as Fine Aggregate.....	85
4.2.4.2	Recycled Concrete as Coarse Aggregate (RCA).....	88
4.2.5	Chemical Admixtures.....	91
4.2.5.1	Air Entraining Admixture (AEA).....	91
4.2.5.2	High Range Water Reducing Admixture (HRWRA).....	91
4.3	Mixture Designs.....	92
4.3.1	Mixture Batching.....	96
4.3.2	Curing.....	97
4.4	Concrete Testing.....	97
5.	Experimental Results.....	99
5.1	General.....	99
5.2	Problems with this Study.....	99
5.2.1	Re-Batch of Mixture #2 (100-RA-C) and Mixture #3 (100-RA-BF) Due to Consolidation Concerns.....	99
5.2.2	Re-Batch of Mixture #2 (100-RA-C) Due to Lack of Specimen Quantities.....	101
5.2.3	Freeze-Thaw Chamber.....	101
5.3	Fresh Concrete Properties.....	103
5.3.1	Slump.....	103

5.3.2	Air Content.....	107
5.3.3	Unit Weight.....	108
5.4	Hardened Concrete Properties.....	111
5.4.1	Compressive Strength.....	111
5.4.2	Permeability.....	124
5.4.3	Freeze-Thaw Durability.....	130
5.4.4	Alkali-Silica Reactivity (ASR).....	149
6.	Conclusions and Recommendations.....	157
6.1	Summary.....	157
6.2	Recommendations for Future Studies.....	161

Appendix

A.	Concrete Mixture Designs.....	169
B.	Material Product Data Sheets.....	176

<u>Bibliography</u>	164
---------------------------	-----

FIGURES

Figure

2.1	Average Compressive Strengths of Concretes Containing GGBFS (Averaged from four cement brands and two aggregate types), (Sippel, 2005).....	16
2.2	Average Compressive Strengths of Concrete Mixtures Containing 70% GGBFS, With and Without HRWRA, (Richardson, 2006).....	20
2.3	Compressive Strengths of RCA Concrete Mixtures Designed with the EMV Method and ACI Method, assumed 28-Day age, Fathifazl et al (2009).....	61
2.4	Average Compressive Strength of RCA Concrete and Control Concrete, Mirjana Malešev et al (2010).....	62
4.1	Average Waste Glass & UCD Sand Gradations.....	87
4.2	Average RCA and UCD Rock Gradations.....	90
5.1	Mixture Slump and HRWRA Type Used.....	106
5.2	Photographs of Failed Compressive Test Specimens, 100-RA-BF (Mixture #3) & 100-RA-C (Mixture #2), Respectively.....	113
5.3	Compressive Strength Curves, 100-RA-BF (Mixture #3).....	114
5.4	Compressive Strength Curves, 100-RA-C (Mixture #2).....	115
5.5	Compressive Strength Curves, 75-RA-BF (Mixture #6).....	115
5.6	Compressive Strength Curves, 50-RA-BF (Mixture #5).....	116
5.7	Compressive Strength Curves, 25-RA-BF (Mixture #4).....	116
5.8	Average Normalized Compressive Strengths, All Mixtures.....	117
5.9	28-Day Compressive Strengths and CDOT Class-D Requirements.....	121
5.10	Photograph of RCPT Test Setup and Apparatus.....	126

5.11	Results of RCPT Testing Performed at Various for All Concrete Mixtures.....	128
5.12	Photograph of Freeze-Thaw Test Specimens, 100-RA-BF (Mixture #3) & 100-RA-C (Mixture #2), Respectively.....	131
5.13	Photograph of Static Frequency Testing Apparatus.....	133
5.14	Photograph of Dynamic Frequency Test Apparatus.....	133
5.15	Average Mass Losses vs. Number of Cycles for All Mixtures.....	145
5.16	Photograph of Preparing ASR Test Specimens, (100% GGBFS Mixture).....	151
5.17	Measuring Length of an ASR Test Specimen.....	152
5.18	Mixture #1 (100% PC) 14-Day Test Results, ASTM C 1567.....	153
5.19	Mixture #2 (50% PC & GGBFS) 14-Day Test Results, ASTM C 1567.....	154
5.19	Mixture #3 (100% GGBFS) 14-Day Test Results, ASTM C 1567.....	154

TABLES

Table

2.1	Mass Percentage of Compounds in GGBFS (Mindess, 2003).....	8
2.2	Average Compressive Strengths for Concretes with Varying GGBFS Contents and Varying Curing Temperatures, (Hale, 2001).....	17
2.3	28-Day Chloride-Ion Penetration Results for Concretes Containing GGBFS vs. Control Mixtures with 100% PC, (Sivasundaram, 1992)..	23
2.4	Visual Results of GGBFS Beams Tested in Sulfate Rich Soil After 5 Years Exposure (Stark, 1989).....	28
2.5	Results of ASR Expansion Tests on GGBFS Specimens Tested According to ASTM-C1260 and ASTM-C1293, (Detwiler, 2003).....	31
2.6	Typical Chemical Compositions Percentages of Soda-Lime Container Glass, (Weihua Jin, 2000).....	37
2.7	Mixture Properties and Slump Results of Concretes Containing Waste Glass as Fine Aggregate, (Shayan et al, 2005).....	39
2.8	Mixture Properties and Compressive Strengths of Concretes Containing Waste Glass as Fine Aggregate, (Shayan et al, 2005).....	43
2.9	Mixture Properties and RCPT Result of Concretes Containing Waste Glass as Fine Aggregate at 380 Days of Age, (Shayan et al, 2005).....	45
2.10	Slump Test Results of RCA Concrete Mixtures, (Malešev, 2010).....	56
2.11	Compressive Strengths of Source Concretes and Concretes Made With RCA From Same Source, (ACI 555R, 2001).....	59
2.12	Table 2.12 Compressive Strengths of Concretes Made with Varying Amounts of CCA, Karthik Obla et al (2007).....	64

2.13	RCPT Test Results of Concretes Made with Varying Amounts of CCA, Karthik Obla et al (2007).....	69
2.14	Average Results of Rapid Freeze-Thaw Testing on Concretes Made with CCA, Karthik Obla et al (2007).....	72
4.1	Colorado Department of Transportation (CDOT) Class-D concrete specifications and field requirements.....	81
4.2	Lehigh Grade 120 GGBFS Physical and Chemical Properties.....	82
4.3	Holcium Type-I-II Cement Physical and Chemical Properties.....	83
4.4	Testing of Recycled Materials.....	84
4.5	Fine Aggregate Properties of Waste Glass and UCD Sand.....	85
4.6	Coarse Aggregate Properties of RCA and UCD Rock.....	88
4.7	Concrete Mixture Design Matrix.....	92
4.8	Fresh and Hardened Concrete Properties Tested.....	98
5.1	Fresh Concrete Properties.....	103
5.2	Measured Unit Weights, Theoretical Unit Weights and Air Contents.....	109
5.3	Measured Unit Weights & Air Adjusted Theoretical Unit Weights....	110
5.4	Average Compressive Strengths.....	112
5.5	Average Compressive Strengths Normalized for 6.5% Air Content...	113
5.6	ASTM C 1202 RCPT Permeability Classifications.....	125
5.7	Results of RCPT Testing Performed at Various Ages for All Concrete Mixtures.....	127
5.8	Mixture #1, CC (Control), Transverse Frequencies & P_c Values.....	135
5.9	Mixture #2, 100-RA-C, Transverse Frequencies & P_c Values.....	135
5.10	Mixture #2 (Re-Batch), 100-RA-C-2, Transverse Frequencies & P_c Values.....	136

5.11	Mixture #3, 100-RA-BF, Transverse Frequencies & P_c Values.....	136
5.12	Mixture #4, 25-RA-BF, Transverse Frequencies & P_c Values.....	137
5.13	Mixture #5, 50-RA-BF, Transverse Frequencies & P_c Values.....	137
5.14	Mixture #6, 75-RA-BF, Transverse Frequencies & P_c Values.....	138
5.15	Mixture #1, CC (Control), Average Dynamic Frequencies & Corresponding P_c Values from Both Specimens.....	139
5.16	Mixture #2, 100-RA-C, Average Dynamic Frequencies & Corresponding P_c Values from Both Specimens.....	139
5.17	Mixture #2 (Re-Batch), 100-RA-C, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens.....	140
5.18	Mixture #3, 100-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens.....	140
5.19	Mixture #4, 25-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens.....	141
5.20	Mixture #5, 50-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens.....	141
5.21	Mixture #6, 75-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens.....	142
5.22	Durability Factors (DF) for Each Specimen, All Mixtures.....	143
5.23	Average Durability Factors (DF) for Each Mixture.....	144
5.24	Mixture Proportions for Testing ASR Potential Based on ASTM C 1567 Procedures.....	150

1. Introduction

Concrete is the most commonly used building material in construction today. Over 2 billion tons of concrete are produced every year and there is a consistent 5% increase each year (ASDSO, 2011). Anything and everything related to construction is open for scrutiny in today's eco-conscience society. People today are more in tune and informed about the negative effects we leave behind for future generations. Green is the new buzz word, and every facet of today's industry is attempting to reduce their carbon footprint. Builders today are under constant pressure to incorporate more and more recycled materials into their products and to become more "earth friendly". The potential use of recycled materials in concrete is a growing interest. Although the use of recycled materials in concrete is not a new advancement, typical replacement values have commonly been on a small order.

Concrete has undergone significant advancements over the years, primarily with regards to cement. Cement is the most important ingredient in a concrete mixture. Cement is the glue of a concrete mixture, binding the aggregates together to form a solid matrix and give the concrete its strength. The cement used in today's concrete is referred to as a "portland cement". The production of portland cement (PC) requires significant amounts of energy. First, the raw materials used to make cement must be acquired and processed. These raw materials must first be quarried from the earth before being crushed and blended to an acceptable level. The blended materials must then be put into a high temperature kiln that transforms these raw materials into clinker. Clinkering is a term used to indicate the stage between sintering and fusion. Sintering indicates no melting takes place, and fusion takes place when 100% of the material is in a molten state. It is estimated that at any given time in the kiln only one quarter of the material is in a molten state. After leaving the kiln this clinker is then processed even further (ground down and mixed with gypsum) before becoming portland cement. The production of portland cement accounts for

5% of the total global CO₂ emissions caused by humans and it is estimated that for every ton (metric ton) of PC produced, an equal ton of carbon dioxide gas (CO₂) is emitted into the atmosphere, (World Buisness Council for Sustainable Development, 2002). Carbon dioxide is a green house gas, and is believed to be a main contributor to global warming. Most of the CO₂ produced comes from the high temperature kilns used in PC production plants. Of all the raw materials used in concrete today, PC is the largest contributor to green house gases.

The use of recycled materials to replace cement content is a common practice and has been for many years. It has been demonstrated that concretes strength, durability and workability can be increased from the use of certain recycled materials. Supplementary cementitious materials (SCM) have been used to replace cement in concrete for thousands of years. Common cement replacements used today are fly ash, silica fume and ground granulated blast furnace slag (GGBFS). Fly ash and silica fume are byproducts of the power and silicon metal industry respectively. GGBFS is a byproduct of the iron manufacturing industry. Fly ash and silica fume are known as pozzolanic materials. GGBFS is a hydraulic cementitious material that also has pozzolanic characteristics. The term pozzolan is used to describe any reactive aluminosilicate material. Pozzolans react with by-products of the cement hydration process in order to develop strength characteristics in concrete. Pozzolans will not typically produce strength alone when mixed with water and therefore require cement within the mixture. For this reason the common thought in the concrete industry is that cement is needed, in at least a moderate amount, to produce strength. GGBFS is also a cementitious material much like cement, and when mixed with water will hydrate and produce strength alone.

Aside from cement, concrete contains several other ingredients. Aggregates have almost always been used in concretes of the past and are always used in today's concretes. Even today's grouts and mortars contain aggregates. Although important in

varying degrees to a concrete mixture, aggregates by and large are a filler material. In fact, it is the bond between the aggregate and cement that is the weakest link in the concrete matrix. Typically it is only in high strength concretes where aggregate strength becomes a contributing factor. Quality aggregate sources are becoming more difficult to find. Many aggregate sources used in the past have been depleted and concrete batch plants are forced to use lesser quality aggregates. To acquire aggregates from the earth, considerable energy must be used to quarry and refine the rock before being suitable for use in concrete. Mining operations are always at the forefront of environmental debate not only from the destructive aspect, but also from an aesthetic standpoint. For these reasons, aggregates are of primary interest with regards to potential replacement with recycled materials.

Many forms of aggregate replacement have been used in the past, from recycled automotive tires and waste metal to pure trash. A less common coarse aggregate replacement that is gaining more interest is recycled concrete aggregate (RCA). Recycled concrete comes from the demolition of buildings, sidewalks, streets, etc. Increasing concern over the potential harmful effects that crushed concrete can have on the environment have caused growing concerns on how to dispose of the materials (due to leaching of chemicals into the watershed), not to mention the increased costs associated with concrete disposal. The diminishing landfill space is a growing concern throughout the world. Reusing crushed concrete is of considerable interest for these reasons.

Another less common aggregate replacement that is gaining more attention is the use of recycled glass as a fine aggregate replacement. Glass containers are typically recycled to make more glass containers, but the potential beneficial use in concrete is gaining interest. Recycling of glass containers is difficult and may be costly if separating the glass into colors is required. Removal of contaminants is also difficult and much of the glass produced today ends up in landfills. In 2007

approximately 13.6 million tons (12.3 metric tons) of waste glass were generated in the United States, and 76% of this glass was disposed of in landfills (Verdugo, 2009). Many states and municipalities have attempted to avoid this difficulty by mandating taxes on glass containers to help push manufacturers into re-using instead of recycling. Whether glass containers are crushed and recycled or re-used, it is estimated that for every ton of glass recycled, 1000 pounds (454 kg) of CO₂ gas is saved from being emitted into the atmosphere, (en.wikipedia.org, 2011).

This research investigates the effects that these recycled materials will have on the fresh and hardened properties of concrete. Several concrete mixtures containing recycled materials were designed and batched for this research. The amounts of recycled materials used in each concrete mixture were varied, and their fresh and hardened properties compared to a control mixture composed of natural (virgin) aggregates and cement. The mixtures developed for this research project had a percentage of aggregates and cement replaced with recycled materials. The natural coarse aggregates were replaced with recycled concrete aggregate (RCA), the natural fine aggregates with waste glass and the cement was replaced with GGBFS. To fully investigate the effects these recycled materials have on the concrete, six mixtures were developed, batched, and tested for structural and durability performance.

The recycled concrete aggregate and waste glasses were fully tested prior to batching. Multiple gradations for both aggregates were completed as well as fineness modulus, specific gravity, absorption capacities and unit weights.

A concrete mixture containing 100% recycled materials (RCA, waste glass and GGBFS) was designed, batched and tested for this research. Three additional mixtures with recycled material contents of 25, 50 and 75% were batched and tested. One mixture was batched that contained 100% RCA and 100% cement as well as a control mixture composed of 100% natural aggregates and 100% cement. All

mixtures were tested for fresh and hardened concrete properties. The fresh concrete properties tested included slump, unit weight and air content. The hardened concrete properties examined were compressive strength, rate of strength gain, permeability, freeze-thaw resistance and alkali-silica reactivity (ASR).

A literature review was performed to investigate any past research completed regarding GGBFS, recycled concrete, and waste glass on the effects each has on concrete's properties. No documented research could be found on the combined effects that all three of these components together have on concrete. This research is therefore considered to be a first of its kind. All testing conformed to ASTM testing standards and when deviated from, notation was made. All data results, details and conclusion of findings of this research are included within this thesis.

2. Literature Review

2.1 Preface

This literature review will not focus on the research that has been done on ordinary concrete containing portland cement and normal aggregates. This review will only focus on the recycled materials used to replace the cement and aggregates. The use of recycled concrete, waste glass and GGBFS will be reviewed. However, no known documented research on the combined effects of these three materials used together in a concrete mixture could be found. Therefore, each material will be reviewed individually in regards to any past research performed.

2.2 Ground Granulated Blast Furnace Slag (GGBFS)

2.2.1 Production

Ground granulated blast furnace slag (GGBFS) is a byproduct of the iron manufacturing industry. It was first commercially produced in Germany in 1853 and slag cements were used to build the underground metro station in Paris in 1889, (PCA, 2005). Slags are residues that come from blast furnace production of various metals and steel, as well as from the production of iron from ore. The slags that come from the steel industry and other metals are not suitable for use in concrete unless they are beneficiated (Mindess, 2003). Typically, the slags that are used in concrete primarily come from the production of iron. During the production of iron, inorganic lime based fluxes are used to remove impurities from the iron. These fluxes rise to the surface of the molten iron and from there, they are removed and cooled. The slags must be cooled quickly (i.e. not air cooled) in order to become a suitable form of hydraulically active calcium aluminosilicate glass. If allowed to dry in air the slag will form inert (non-reactive) calcium magnesium silicates (Mindess, 2003). The

process of “quenching” (using water to cool) is used to cool the slags. The method of cooling varies from one plant to another and the amount of water used will have a direct impact on the quality and strength of the slag (Bureau of Reclamation, 1988). Granulation and pelletization are two ways in which the slag can be quenched. The granulation process is used to produce GGBFS. During the granulation process the molten slag is broken up by jets of water before being immersed in a water bath. This process produces small 4 mm-sized granules which are then ground down to cement fineness. Pelletization produces larger granules which are used in concrete as light weight aggregates.

2.2.2 Physical, Chemical and Reactive Properties

There are three ASTM specified grades of GGBFS produced today, Grade 80, Grade 100 and Grade 120. This classification is called the “slag-activity index” and applies to the GGBFS ability to improve the compressive strength of mortar cubes mixed with 50% cement and 50% GGBFS when compared with reference cubes containing 100% cement (ASTM C 989). The 28 day compressive strength of Grade 80, Grade 100 and Grade 120 cubes must be at least 75, 95 and 115% of the strength of reference cubes when compared respectively. GGBFS is usually ground down to a fineness that exceeds normal portland cement to increase reactivity. Blaine fineness ranges are anywhere from 3500 cm^2/g to 8000 cm^2/g . The fineness of the slag varies with grade and will therefore also affect the fresh concrete properties as well. The Blaine fineness of Grade 120 slag is higher than Grade 100, which in turn is higher than Grade 80 slag. The higher the fineness the higher the water demand. Ultimately, a concrete made with Grade 120 slag will have a lower slump value than a similar mixture made with Grade 80 slag.

The chemical composition of GGBFS used in concrete today varies little from plant to plant due to the fixed process of steel making and is not a significant concern (Hooten, 2000). GGBFS is rich in lime, silica and alumina. The typical mass compound percentages are shown below in Table 2.1. The amounts of sulfur are limited to 2.5% as sulfide and 4.0% as sulfate per ASTM C 989. Residual iron is also present as ferric oxide.

Table 2.1 Mass Percentage of Compounds in GGBFS (Mindess, 2003)

Compound	Percentage	Compound	Percentage
CaO	35 - 45	MgO	5 - 15
SiO ₂	32 - 38	Fe ₂ O ₃	< 2
Al ₂ O ₃	8 - 16	Sulfur	1 - 2

When hydraulic cement (PC or GGBFS) is mixed with water, a chemical reaction takes place known as “hydration”. Equations 2.1 and 2.2 below show the generalized hydration process of tri-calcium silicate (C₃S) and di-calcium silicate (C₂S) in cement (ceramic notation), (Mindess, 2003).



Where: “C₃S₂H₃” is referred to as Calcium Silicate Hydrate (C-S-H)

“CH” is referred to as Calcium Hydroxide

The calcium silicate hydrates are the primary strength producing compounds in a concrete mixture. Tri-calcium silicates typically form early in the hydration

process and give the concrete early age strength. The di-calcium silicates primarily form much later and will contribute more to later age strengths. The hydration of slag is much slower than a typical PC hydration and may take several months depending on grade to reach equivalent 28 day strengths when compared with concretes made with pure cement as the binder. The cause of this slow hydration is believed to be due to impervious coatings of amorphous silica and alumina that form around the slag particles during the first stages of the hydration process (Mindess, 2003). It is believed that GGBFS hydration primarily produces C_2S as opposed to a PC hydration which primarily produces C_3S (ASTM C-989). The hydration of GGBFS is also alkali activated and the hydration rate will depend on the alkali content of the cement and the alkali content of the slag.

The calcium hydroxide (CH) produced during the hydration process is not considered a main strength producing compound, may actually cause durability problems in a hardened concrete. CH in a hardened state is water soluble and crystalline in structure. Leaching of CH from brick mortars can be readily seen as well as from concrete (chalky white substance). This leaching process is called efflorescence. Calcium hydroxide is also a component in alkali-silica reactions as well as sulfate attack. Although CH is not a main strength producing component of a cement hydration process, it is a primary ingredient for a pozzolanic reaction. ASTM C 618 describes a pozzolanic material as “a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties”. GGBFS contains amorphous silica and will react with calcium hydroxide to form calcium silicate hydrates. Equation 2.3 below shows the generalized pozzolanic calcium hydroxide and silica hydration process (ceramic notation), (Mindess, 2003).



Due to this conversion of calcium hydroxide to calcium silicate hydrate, a concrete's strength and durability can be greatly increased by the use of pozzolanic materials.

2.2.3 The Effects of GGBFS on Fresh Concrete Properties

2.2.3.1 Slump

A majority of past research indicates the effects of GGBFS on concrete's workability are favorable (i.e. increased workability). According to a report issued by ACI regarding the use of GGBFS in concrete mixtures, typically the addition of GGBFS in a concrete mixture will improve workability and increase slump, (ACI 233R, 2000). This is believed to be caused by the texture and density of finely ground blast furnace slag particles, which have smooth glassy surfaces and have a denser outer surface than cement particles. GGBFS particles will therefore require less water immediately after mixing than cement particles which absorb water rapidly (ACI 233R, 2000). According to Wang Ling et al (2005), the increase in workability observed with concrete containing GGBFS may be a result of an increased repulsion potential between the cement and GGBFS particles. They also attributed this increase in workability to the larger specific area of GGBFS particles.

Osborne (1989) investigated the effects of GGBFS on slump, compaction factor and Vebe for concrete containing 0, 40 and 70 percent replacements with slag. He found that as the percentage of GGBFS increased, the ratio of water to cementitious materials needed to be reduced in order to maintain workability properties similar to the control mixture having only cement as the binder. Research conducted by the Virginia Department of Transportation (1999) showed that it is

possible to reduce the w/c (water to cement ratio) of a concrete mixture containing GGBFS to achieve the same slump as a mixture containing only cement due to the decreased water demand. Contrary to the previous work cited, the Missouri Department of Transportation (2006) found that average slump values for a 70% GGBFS mixture (Grade 120) were 2 inches (51 mm) less than that of control mixtures having only PC as the binder and similar w/c. Similar results were reported by Hale (2001) who investigated two GGBFS mixtures with 25 and 50% replacements. Both mixtures had significantly less slump than the control mixture with only cement. This was attributed to the higher fineness of the Grade 120 slag when compared to the fineness of the cement.

2.2.3.2 Air Content

Information regarding the effects of GGBFS on air content was limited. Much research was found regarding fresh concrete properties on concretes mixed with GGBFS and air entraining admixtures however air content was rarely discussed. According to the Portland Cement Association (2005), ground slags will have a varying effect on the required dosage rates of air-entraining admixtures. ACI -233R (2000) indicates that typically the required amount of air-entraining admixture will increase if the GGBFS is finer than the cement.

Sivasundaram and Malhotra (1992) studied the effects of high volume GGBFS concrete with target air contents of 5%. When studying the effects of air entraining admixtures (AEA), they found that as GGBFS content increased from 60 to 75%, the amount of AEA needed almost doubled. When comparing GGBFS mixtures to the control mixture with 100% cement, in some cases the AEA dosage required more than quadrupled. They also found that if slag content was held constant and cement content adjusted the effects on AEA dosage remained constant, which

would indicate the slag content is the governing factor. Richardson (2006) investigated the air void system characteristics of several concrete mixtures composed of 70% GGBFS replacement and found that the results were similar to control mixtures made with only PC as the binder.

2.2.3.3 Time of Set

In general, research has shown that initial and final set times will be effected by the use of GGBFS. According to ACI-233R (2000), the time of set for concretes incorporating GGBFS will be increased and dependent on the initial temperature, replacement amount, cement characteristics and w/c. The time to initial set for fresh concrete temperatures around 73 °F (23 °C) will typically be one half to one hour longer and for temperatures above 85 °F (30 °C), no significant changes were observed when compared to a mixture with 100% cement. Significant retardations have been observed at lower temperatures but these effects can be reduced by the use of conventional accelerators such as calcium chloride.

Hogen and Meusel (1981) found that at higher temperatures (above 85° F) the initial and final set times were similar to those of normal concrete with 100% cement. Results were similar for slag fineness ranges between 4500 cm²/g and 6000 cm²/g (2197 ft²/lb and 2930 ft²/lb) which give an indication that slag fineness is not related to set time. Sakai et al (1992) found that only the final set times increased when comparing normal concrete and GGBFS concretes. The initial set times were not affected. They also used varying slag finenesses between 3000 cm²/g to 6000 cm²/g (1465 ft²/lb and 2930 ft²/lb) which strengthens the belief that slag fineness is not a factor in set time.

2.2.3.4 Temperature

The temperature rise in concrete is caused by the hydration process and is of considerable interest in mass concrete placements. This rise in temperature can take place over many days, months or even years depending on the size of placement. Excessive temperatures can lead to thermal induced cracking, especially in mass placements. The use of mineral admixtures such as fly ash and GGBFS in mass concrete have essentially eliminated the need for production of Type IV cement (low heat of hydration) in the United States. Since the reaction rate of GGBFS is much slower, the heat is released gradually over longer periods, and the temperature of the concrete is reduced (Mindess, 2003). According to ACPA, the decrease in temperature is inversely proportional to the fineness, i.e. the finer the GGBFS the less of a decrease in temperature due to the higher reactivity of finer slags (www.ndconcrete.com).

The Bureau of Reclamation (2010) performed adiabatic temperature studies on several concrete mixtures containing various pozzolans and Type II low heat of hydration cement. The pozzolans used were a Class F fly ash and a Grade 120 GGBFS. Three mixtures containing 30% Class F fly ash and two mixtures containing 70% GGBFS were batched and tested. The total cementitious contents for the fly ash and GGBFS mixtures were approximately 500 lb/yd³ and 375 lb/yd³ (297 kg/m³ and 222 kg/m³) respectively. The temperature rise for the GGBFS mixtures was approximately 55 °F (13 °C) over the course of 56 days. The average temperature rise for the fly ash mixtures was 73 °F (23 °C) over the course of 56 days (high of 80.1 °F (26.7 °C) and low of 68.8 °F (20.4 °C)). This indicates a significant temperature decrease between 14 °F and 25 °F (-10 °C and -4 °C) when comparing the GGBFS mixtures to the fly ash mixtures. This decrease in temperature rise may also be due to the decreased cementitious content of the GGBFS mixtures.

Sivasundaram and Malhotra (1992) investigated the autogenous temperature rise and peak temperatures for concretes containing 50 to 70% GGBFS. They found that high volumes of slag content would suppress the autogenous temperature rise of concretes. In addition, they found that with increasing slag content the peak temperature rise decreased as well.

2.2.4 The Effects of GGBFS on Hardened Concrete Properties

2.2.4.1 Strength

Studies on the effects of GGBFS on strength are varied. Typically, the early age strengths of concretes containing GGBFS will be lower and later age strengths higher when compared with normal PC concretes. The rate of strength gain appears to be dependent on the grade of GGBS used in the mixture as well as the replacement percentage. When compared to a normal cement concrete, mixtures with Grade 120 slag show decreased early age strengths and increased later age strengths (beyond 7-days). If Grade 100 slag is used the general trend is lower early and mid age strengths (1-21 days), and similar or greater strengths at later ages. Grade 80 slag will typically lower all strengths at both early and later ages (ACI 233R, 2000). Other factors also affected the strengths, such as the use of water reducing admixtures and curing regimens (particularly temperature). There also appears to be ideal replacement percentages where the greatest strength gains are realized.

Hogan and Meusel (1981) investigated the compressive strengths of GGBFS concrete for a variety of replacement percentages. The GGBFS contents examined were 40, 50, and 65 percent. These mixtures were compared to a control mixture consisting of cement as the only binder. Two w/c ranges were used for air-entrained mixtures and non-air-entrained mixtures. The w/c was 0.40 and 0.55 for air-entrained

mixtures and 0.38 and 0.55 for non-air-entrained mixtures. All specimens were moist cured. The fineness of the slag used ranged from 4500 cm²/g to 6000 cm²/g (2197 ft²/lb to 2930 ft²/lb). The results of the compressive strength tests show that the GGBFS concretes gained strength at a slower rate during the early ages (1-7 days) than the control mixture made with 100% cement as the binder. Between 7 and 10 days the strength gains of GGBFS concretes were greater than the control mixture. When comparing 28-day strengths for all mixtures the concrete made with 40% GGBFS, w/c of 0.38 and no air-entrainment had the highest strength of 8220 psi (56.7 MPa) compared with the control having a 28-day strength of 7240 psi (50.0 MPa). For all mixtures, regardless of air-entrainment or w/c, the 28-day strengths decreased as slag content increased from 40% to 65%. For non-air-entrained mixtures, increasing the slag content from 40% to 50% had only a minimal effect on the compressive strength at 28-days. When the slag content was further increased from 50% to 65%, the 28-day strengths decreased significantly by as much as 24%. All mixtures incorporating GGBFS except the mixture with w/c of 55% and non-air-entrained, exceeded the 28-day strengths of the similar control mixtures

Cramer and Sippel (2005) investigated concretes incorporating 30 and 50% replacements with GGBFS and two different aggregate sources and four different cement brands. The aggregates used were a limestone aggregate and an igneous aggregate. Type I cement and Grade 100 GGBFS were used for all mixtures. All mixtures were air-entrained and had air contents of 6%. They found that regardless of replacement amount, most GGBFS mixtures had lower strengths than control mixtures at all ages (with a few exceptions). Figure 2.1 shows the average compressive strengths for the mixtures based on GGBFS replacement.

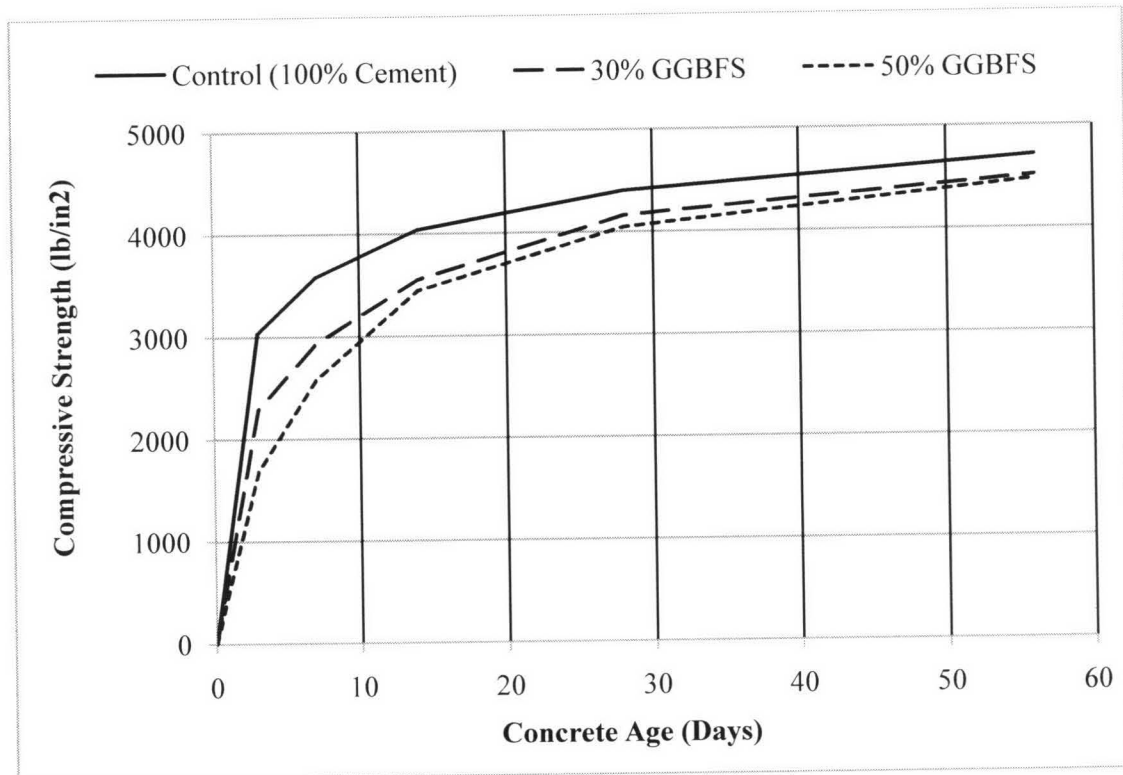


Figure 2.1 Average Compressive Strengths of Concretes Containing GGBFS (Averaged from four cement brands and two aggregate types), (Sippel, 2005).

Although not shown on Figure 2.1, the 365-day strength trend was similar and the control mixture on average had the highest strength. This is contrary to typical trends where Grade 100 GGBFS concretes exhibit higher later age strengths (beyond 21 days) than normal concrete (ACI, 2000).

Hale (2001) investigated the effects of GGBFS and curing regimens on concrete. The cement used was a Type I and the GGBFS was a Grade 120. The total cementitious content for all mixtures was 658 lb/yd³ (390 kg/m³). Normal aggregates were used as well as a w/c of 0.39 for all mixtures. An air-entraining admixture was used for all mixtures to achieve a target air content of 7%. Three different curing

regimens were followed; standard conditions of 73 °F (23 °C) and 100% humidity, cold weather conditions of 50 °F (10 °C) and 100% humidity, and hot weather conditions of 83 °F (28 °C) and 100% humidity. A replacement percentage of 50% GGBFS was examined with standard and hot curing conditions, and 25% for cold, standard and hot curing conditions. All mixtures were compared to a control mixture consisting of 100% cement. Table 2.2 shows the results from this study.

Table 2.2 Average Compressive Strengths for Concretes with Varying GGBFS Contents and Varying Curing Temperatures, (Hale, 2001).

Mix ID	Curing Temp.	3-Day	7-Day	28-Day	56-Day	90-Day
Control	70 °F	2820	3500	4510	4700	5090
25% GGBFS	70 °F	3100	3940	5820	6640	7040
50% GGBFS	70 °F	3050	4310	6390	7150	7670
Control	50 °F	1200	2520	3700	3980	4560
25% GGBFS	50 °F	1500	2900	5140	5840	6100
Control	83 °F	3760	4380	5460	6070	6380
25% GGBFS	83 °F	3670	4990	6610	7110	7200
50% GGBFS	83 °F	3470	5500	6780	7570	8660

What is interesting in this study is that both the 25 and 50% replacement mixtures exceeded the control mixtures strength at nearly all ages and curing regimens. The only exception to this trend was for the hot cured specimens at 3-days in which the control specimen had a strength of 3760 psi (25.9 MPa) and the 25 and 50% specimens had strengths of 3670 psi and 3470 psi (25.3 MPa and 23.9 MPa)

respectively. Yet after the 3-day mark, both GGBFS specimens rebounded and overtook the control specimens during the hot cured regimen. As expected, the hot curing regimen increased compressive strengths for all mixtures and was attributed to the evaporation of water in the fresh concrete state which can lead to a reduced w/c. As expected, the cold curing regimen decreased both specimens strength throughout the range when compared to the standard curing temperature regimen. As stated in the report, the air contents for the 50% GGBFS mixture were one to two percent less than the other mixtures. Another interesting point is that the air content for the control mixture cured under the cold regimen was 9.1%, which was 3.0% higher than the 25% GGBFS mixture which had an air content of 6.1%. These results do not indicate whether or not the compressive strengths were normalized for air content and therefore it assumed they were not.

Sivasundaram and Malhotra (1992) investigated the effects of GGBFS on concretes using high replacement amounts between 50 and 75%. The w/c ranged from 0.27 to 0.45. All mixtures contained air-entraining admixture and had target air contents of 5.0%. A high range water reducing admixture (HRWRA) was also used on all mixtures. All specimens were cured at 100% humidity and 68 °F (20 °C). The cement used was a Type I, however the GGBFS grade nor the fineness was specified. The total cementitious contents varied across the board (from 242 to 428 lb/yd³ (144 to 254 kg/m³)) and when coupled with the varying w/c, the results are difficult to interpret. They found that in all mixtures with 169 lb/yd³ of total cementitious, the strengths for all ages increased when slag content increased from 50 to 70 and 75%. They also determined that increasing the slag content from 70 to 75% does not yield any significant benefits to strength. They found that for cementitious contents of 169, 211 and 253 lb/yd³ (100, 125 and 150 kg/m³) the optimum slag replacements were 70, 65 and 55% respectively. This indicates that as total cementitious contents increase, the benefits of slag content decrease. In general, they determined that slag concretes

typically attain most of their strength by 28 days when compared to control mixtures with only PC. All slag concretes were equal to or had higher strengths than control mixtures beyond 7-days.

Richardson (2006) investigated two concrete mixtures with GGBFS contents of 70% of the total cementitious materials. A HRWRA was used on only one GGBFS mixture to study the effects of using a HRWRA on a slag concrete. A control mixture was batched with a Type I cement while a Type II cement was used for the two GGBFS mixtures. The type of slag used for both mixtures was a Grade 120. Normal aggregates were used and average w/c values ranged from .399 to .419. Air entraining admixtures (AEA) were used to achieve target air contents of 6.0% on all mixtures. The average results of compressive tests performed on three mixtures each for the control, 25 and 50 GGBFS mixtures are shown in Figure 2.2.

The results of this study showed that both 70% GGBFS mixtures had significantly less strengths for all ages when compared to the control mixture. The strength gain for early age (up to 7-days) was lowest for the 70%GGBFS when compared to the 70%GGBFS-HRWRA and control mixture. The strength gain for the HRWRA mixture was greatest between 7 and 28-days. The effects of using the HRWRA on GGBFS mixtures can readily be seen. Beyond 7-days the HRWRA mixture had greater strengths than the GGBFS mixture without HRWRA. This is no surprise however since the use of HRWRA creates a superior microstructure within the concrete matrix. The control mixture only had slightly higher air contents (around 1.0% higher) than both the GGBFS mixtures and therefore air content was probably not a cause of the higher compressive strengths for the control mixture. A major cause for the slower strength gains for both GGBFS mixtures is probably due to the use of Type II cement. The use of Type II cement will typically lead to slower strength gains caused by the lower heat of hydration. Although not shown in Figure 2.2, the 365 day strengths show similar trends with the highest strength of 6970 psi

(48.1 MPa) coming from the control, and strengths of 5645 and 5010 psi (38.9 and 34.5 MPa) from the 70%GGBFS-HRWRA and 70%GGBFS mixtures respectively. If similar cement types were used in all mixtures, the results would most certainly be different and the GGBFS mixtures may have been on par with the Control mixture, especially at later ages.

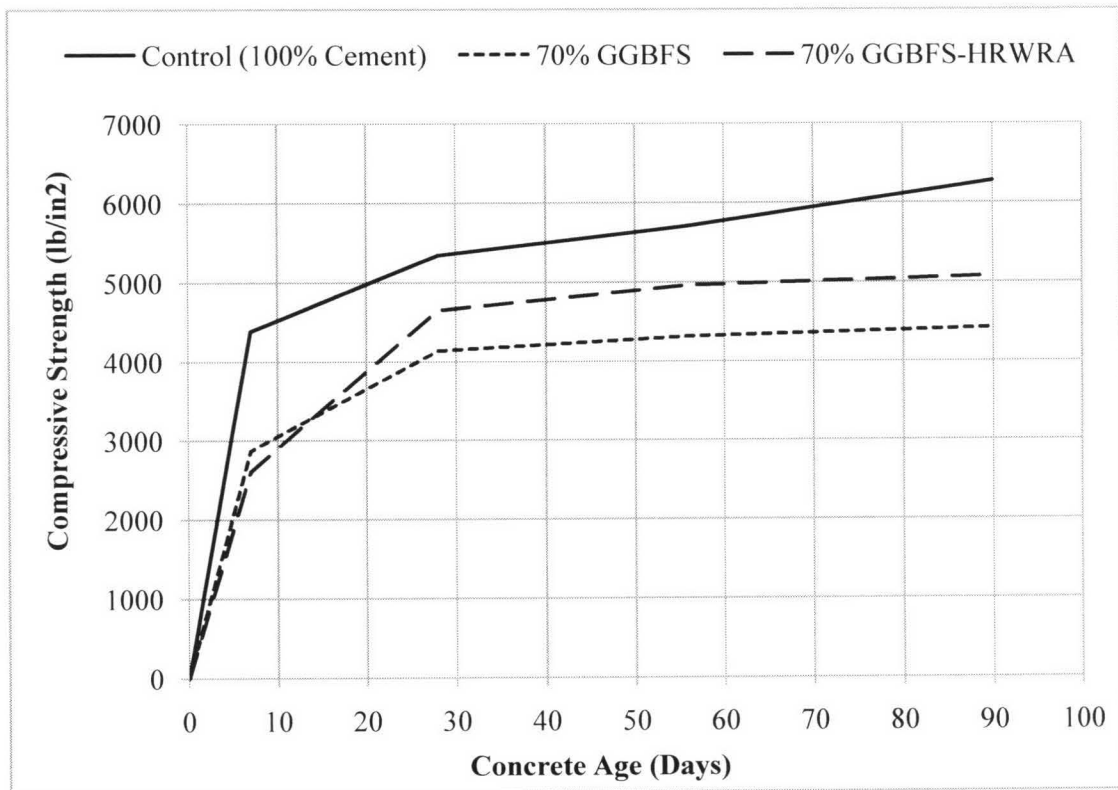


Figure 2.2 Average Compressive Strengths of Concrete Mixtures Containing 70% GGBFS, With and Without HRWRA, (Richardson, 2006).

2.2.4.2 Permeability

Most studies of permeability on concretes containing GGBFS have shown that permeability decreases with increasing slag content (ACI, 2000). A concrete's permeability depends on the interconnectivity of the pores within a hardened concrete. The less connected these pores are, the more difficult it is for water and chemicals to penetrate. These pores can vary in size from the microscopic to those that can be seen by the naked eye. The use of GGBFS not only eliminates pores but is also believed to reduce the size of pores as well. This is due in part to the transformation of CH within a pore to C-S-H during a pozzolanic reaction. In essence, this process decreases the solubility of the concrete (Mindess, 2003). Another important factor on a concrete's permeability is the w/c. The lower the w/c the less permeable the concrete will be.

There are three categories of measurement used to determine a concrete's permeability. Two of these are traditional methods which involve the flow or movement of water through a concrete, while a third indirect method involves the flow or movement of an electrical current. The indirect method is the most commonly used due to the difficulty in calculating the flow of water through a concrete medium. The indirect methods are known as rapid chloride-ion permeability tests (RCPT). In these tests a voltage is applied to the sides of a concrete specimen (typically a cylinder 4 inch (102 mm) diameter by 2 inch (51 mm) height with solutions of sodium chloride and sodium hydroxide on opposite ends. The amount of charge that has passed through the specimen in 6 hours is used as a measurement of permeability. Regardless of method, permeability measurements show significant variability with coefficients of variation between 30 and 50% (Mindess, 2003). This has caused some researchers to believe that the RCPT method only measures concretes conductivity and not the permeability.

According to Shi, et al, (1998), who investigated the effects of GGBFS on the permeability of concrete, the addition of mineral admixtures such as fly ash, silica fume and GGBFS may affect the RCPT values due to the altering of the conductivity. This change in conductivity is caused by the change in pore fluid chemistry. A hardened concrete's pore fluid typically consists of sodium (Na), potassium (K) and hydroxide (OH) ions. The report indicated that the introduction of GGBFS decreased the concentration of the K and OH ions while increasing the Na ion concentration. With these changes in pore chemistry comes a change in a concrete's specific conductivity. When comparing the specific conductivity of a concrete made from 100% PC to a mixture made with 50% GGBFS, the difference in specific conductivity at 28 and 730 days of age was 3.25 and 24% respectively. They concluded that RCPT tests should not be used to evaluate the permeability of concretes containing mineral admixtures.

Sivasundaram and Malhotra (1992) studied the effects of GGBFS on a concrete's permeability using high replacement amounts between 50 and 75%. The w/c ranged from 0.27 to 0.45. The permeability was determined based on the specimens resistance to chloride-ion penetration measured according to AASHTO T277-83. All mixtures contained air-entraining admixture and had target air contents of 5.0%. A high range water reducing admixture (HRWRA) was also used on all mixtures. All specimens were cured at 100% humidity and 68 °F (20 °C). The cement used was a Type I; however neither the GGBFS grade nor the fineness was specified. The chloride-ion penetration tests were performed at 28 days of age. The results of this study are shown in Table 2.3.

Table 2.3 28-Day Chloride-Ion Penetration Results for Concretes Containing GGBFS vs. Control Mixtures with 100% PC, (Sivasundaram, 1992).

Mixture Series	w/c	GGBFS Replacement	Chloride-Ion Penetration
<i>(#)</i>	<i>(ratio)</i>	<i>(%)</i>	<i>(Coulomb)</i>
3	0.29	75	174
2	0.34	70	230
6	0.28	70	213
9	0.27	65	321
1	0.45	60	829
5	0.36	60	325
8	0.30	60	276
4	0.45	50	1159
7	0.38	50	383
Control-1	0.39	0	2984
Control-2	0.31	0	1283
Control-3	0.27	0	1305

The result show that the resistance of the concretes to chloride-ion penetration increased with increasing slag content. Mixture #6 and #8 had chloride-ion penetrations of 213 and 276 coulombs respectively. Compared with similar w/c control mixtures #3 and #2 which had a chloride-ion penetration of 1305 and 1283 coulombs, the decrease in penetration is roughly between 85 and 80 percent. What is also easily verified from this research is that as w/c decreases, the chloride-ion penetration resistance increases substantially. When comparing the control mixtures and GGBFS mixtures with two or more w/c values, as w/c decreased the penetration decreased.

2.2.4.3 Freeze-Thaw Durability

The effect of GGBFS on a concrete's resistance to freeze-thaw cycles is scattered. Some studies have found little to no benefit from the use of GGBFS, while others have found detrimental effects from the use of slag. Several factors determine how well a concrete will withstand freeze-thaw cycles. A concrete's resistance to freezing and thawing depends on the permeability, the degree of saturation of the paste, the amount of freezable water, the rate of freezing, the average distance from any point in the paste to a free surface where ice can safely form and strength (Mindess, 2003). Air-entrained concrete is commonly used today in areas where freeze-thaw durability is needed. These air-entrained concretes have small evenly dispersed air pockets which allow ice crystals to form (within the free surface) in the paste without causing excessive tensile forces. However, even air-entrained concrete will not be able to withstand repeated freeze-thaw cycles without strength. Strength and w/c go hand in hand and according to theory a fully hydrated concrete with w/c below 0.36 will not even need air-entrainment. This is due to the lack of freezable water available within the paste. It has also been noted that current test methods to determine a concrete's resistance to freezing and thawing such as ASTM C666, are not clear indicators to actual field conditions and the results should be used with caution (Mindess, 2003). Laboratory conditions caused by ASTM C666 are extremely harsh and unrealistic for most applications, especially the high rate of freezing (5°F/h (2.8°C/h)).

Hogan and Meusel (1981) investigated the durability of concretes containing 50% GGBFS using ASTM C666 procedure A. Concrete beams were cast for a mixture containing 50% GGBFS as well as concrete made from 100% PC as a control. The total number of freeze-thaw cycles these beams were subjected to was 301. The results indicated that both concretes achieved similar results. The 50% GGBFS had a durability factor of 91 and the control mixture had a durability factor of 98. They also concluded that the weight losses and expansion differences between the

mixtures determined during testing were negligible and both mixtures produced sound concretes.

Lane and Ozyildirim (1999) performed freeze-thaw testing on several concrete mixtures containing 25, 35, 50 and 60% GGBFS replacements. The GGBFS was Grade 120 and all mixtures had w/c of 0.45. The testing method used was ASTM C666 procedure A, where the specimens are subjected to 300 cycles of freezing and thawing while being immersed in a 2% sodium chloride by mass solution. This procedure is quite severe and all specimens performed very well with durability factors greater than 100 for all specimens including the control beam made with 100% PC. The 25, 35 and 50% GGBFS beams had percentage of weight losses below 3% while the 60% GGBFS beam had a loss of 7.7% which exceeds the maximum permissible limit of 7.0%. The scaling observed on all GGBFS specimens was only on the surface and it is believed this would not be detrimental to use on roadways (a primary basis for this study) since the loss of the top surface lends to increased traction for tires.

2.2.4.4 Resistance to Sulfate Attack

Sulfate attack is a complex process which may involve all hydration products within the cement paste. However, a clear correlation has been established between the tricalcium aluminate (C_3A) content and a concrete's susceptibility to sulfate attack. When C_3A hydrates with Gypsum (added to ground clinker during cement production) and water, the product formed is called ettringite. Ettringite is stable as long as gypsum is present. However, when the gypsum is used up, ettringite and any remaining non-hydrated C_3A will react a second time to form mono-sulfoaluminate. It is the mono-sulfoaluminate that is very unstable in the presence of sulfate. Sulfates are present in many ground waters, soils and clays and also in sea water. If sulfate

comes into contact with mono-sulfoaluminate, a reaction will take place and ettringite will reform causing excessive stresses within the cement paste leading to rapid deterioration. Reactions can also take place between sulfates and the CH and C-S-H within a hardened cement paste. Reducing a concrete's susceptibility to sulfate attack involves decreasing the C_3A content, lowering the w/c which increases the strength and decreases the permeability.

It is well documented that concretes containing GGBFS replacements of 50% or more have shown significantly greater resistances to sulfate attack. A minimum replacement of 50% GGBFS is recommended when used with Type I cements that have C_3A contents up to 12% (ACI 233R, 2000). It is also recommended that the alumina content of the GGBFS be no greater than 11% and in certain studies it was found that when slags are used that have less than 11% alumina, increases in sulfate resistance were found regardless of C_3A content in the cement. Hogan and Meusel (1981) investigated the resistance to sulfate attack with concretes having between 40 to 65% replacements with GGBFS. The method of testing used was the Wolochow Method (lean mortar bar method). They found that mortar bars with 65% GGBFS had expansions of only 0.07% at 70 weeks of age compared to 0.12% for similar bars made with 100% PC.

Frearson and Higgins (1992) conducted research on mortar prisms to determine the effects of GGBFS on resistance to sulfate attack. The prisms were made with Type I cement and varying amounts of GGBFS up to 70% replacement. Neither the grade of slag nor the fineness was specified in the report. The specimens were cured in water for a period of 14 days. After curing the specimens were placed and kept in a 0.31 molar solution of sodium sulfate (Na_2SO_4) for 4 years. It was determined from this research that as the GGBFS content increased the expansive damage caused by sulfate attack decreased. The prism made with 70% GGBFS had a

total expansion less than 0.01%, compared to the control prism made of 100% PC which had completely disintegrated after only 4 months of exposure.

Stark (1989) performed an extensive study into the effects of sulfate rich soils on concrete. A total of 48 different concrete mixtures were tested during this research and included three types of cement (I, II and V) as well as various percentages of supplementary cementitious materials including GGBFS. Two different GGBFS types were used, however the grade was not specified. The fineness of these two slags were 5485 and 4415 cm^2/g (2678 and 2156 ft^2/lb). Three separate total cementitious batches with contents of 376, 517 and 658 lb/yd^3 (223, 307 and 390 kg/m^3) for each mixture were investigated. Three beams measuring 6 inch x 6 inch x 30 inch (152 mm x 152 mm x 762 mm) were made for each mixture and cured for 27 days in 100% humidity and 73 °F (23 °C) and thereafter for the remainder of one year at 50% humidity and 73 °F (23 °C) before testing began. Cylinders were also cast for testing the 28-day strengths of the mixtures. After one year of curing the specimens were placed outside three inches deep in sulfate rich soil for a total of 5 years and inspected on an annual basis. Specimens were rated on a scale of 1 to 5 based on amount of deterioration, where 5 is the worst rating and 1 indicates no deterioration. A summary of the results including the control specimens and specimens made with GGBFS are shown in Table 2.4.

In general the results show a significant reduction in sulfate resistance as w/c increases. This is most probably due to the decrease in permeability as w/c increases. All GGBFS beams with cementitious contents of 376 lb/yd^3 (223 kg/m^3) performed poorly with the best rating of 4.6. Results indicate that conditions only slightly improved for the GGBFS mixtures as cementitious contents increased to 517 and 658 lb/yd^3 (307 and 390 kg/m^3), however the w/c also decreased substantially as well. Regardless of GGBFS replacement, all control beams made with 100% PC (regardless of cement type) outperformed the GGBFS beams in nearly all the tests.

When comparing the 40% replacement to the 65% replacement GGBFS beams, the increase in slag content decreased the resistance to sulfate attack. What is also interesting is that when comparing the control beams made with 100% Type II PC to the GGBFS beams made with Type II cement, the GGBFS had a negative effect. The alumina content of both these slags was 9.40 and 6.1% which is below the maximum recommended amount (11%) specified by ACI-233R to achieve adequate sulfate resistance.

Table 2.4 Visual Results of GGBFS Beams Tested in Sulfate Rich Soil After 5 Years Exposure (Stark, 1989).

Cement Type	Mix ID	SLAG Fineness	376 lb/yd ³			517 lb/yd ³			658 lb/yd ³		
			w/c	Strength	Rating	w/c	Strength	Rating	w/c	Strength	Rating
		(cm ² /g)	(#)	(psi)	(1-5)	(#)	(psi)	(1-5)	(#)	(psi)	(1-5)
I	Control	N/A	0.71	4030	5.0	0.46	6380	4.3	0.39	7440	1.5
II	Control	N/A	0.71	3980	3.9	0.49	5640	2.9	0.38	7120	1.7
V	Control	N/A	0.65	4140	4.4	0.45	5420	1.8	0.37	6620	1.3
II	40% GGBFS	5485	0.67	4900	5.0	0.47	6750	3.4	0.39	7400	2.0
II	65% GGBFS	5485	0.66	4230	5.0	0.47	6180	4.0	0.37	6900	3.0
II	40% GGBFS	4415	0.71	3690	4.6	0.50	6080	3.3	0.41	7620	3.1
II	65% GGBFS	4415	0.72	3060	4.8	0.50	5290	3.7	0.40	7000	3.4

The author pointed out that these results are contrary to common laboratory results regarding the sulfate resistance of GGBFS concretes, and this was attributed to the harsher conditions of this test, albeit more realistic. Specimens were outside and underwent constant wetting and drying cycles. This is opposite of laboratory conditions where specimens are submerged in a sodium sulfate solution for the duration of the test. It was also pointed out that almost all specimens were only deteriorated above grade, and the bottoms of most beams were mostly undamaged. This was probably due to the more consistently moist environment below the soil (i.e. less wetting and drying cycles). This strengthens the belief that laboratory conditions do not clearly represent actual exposure conditions where concrete is in contact with sulfate rich soils.

2.2.4.5 Alkali-Silica Reactivity (ASR)

Studies have shown that the incorporation of GGBFS in concrete can reduce the potential for ASR when reactive aggregates are used. In general, the causes of alkali-silica reactions are due to the alkali content in cement and the silica in the aggregate. The reaction between a reactive aggregate and the alkalis from the cement can cause an expansive gel to form around the aggregate particle. This expansion can ultimately destroy a concrete. ASR is a complicated reaction which involves many different factors. The general factors that can be used to control the effects of ASR in a concrete are: control of the alkali concentrations (primarily from cement), control of the amount of reactive silica (aggregate), control moisture penetration (permeability), control the pH in the pore solution and alteration of the alkali-silica gel that forms during the reaction (delay deterioration), (Mindess, 2003). GGBFS uses alkali to hydrate and thus helps reduce the alkali content available to react with aggregates. GGBFS also helps reduce permeability which helps reduce the ASR potential.

Simply finding aggregates that are not reactive is becoming more and more difficult due to the availability of quality aggregate sources. As previously stated in the introduction, the availability of quality aggregate sources has diminished over time and concrete batch plants are being forced to use lesser quality aggregates. Therefore, other techniques are being used today to eliminate the potential for ASR. The use of pozzolans is a common method employed to reduce the potential for ASR in concrete. According to ASTM-C989, concretes made with GGBFS replacements greater than 40% showed reduced expansions due to alkali-silica reaction when used with cements having alkali contents up to 1.0%. According to ACI-233R, replacements of 50% or more with GGBFS have been effective in reducing the potential for alkali-silica reactions when used with high alkali cement and reactive aggregates.

Detwiler (2003) studied the effectiveness of using GGBFS in mitigating the expansive reactions caused by ASR. The purpose of the research was to investigate the effectiveness that pozzolans have on three particular factors that are believed to be connected with use of pozzolans in controlling ASR. These factors included; dilution of the alkalis in the cementitious paste, reduction of permeability and binding of the alkalis. A moderately reactive aggregate was used as well as two different Type I cements having alkali contents of 0.46 and 0.92%. Two GGBFS specimens were made for each test and consisted of 35 and 50% replacements. Multiple specimens were also made with fly ashes having varying CaO contents. Control specimens were also made with 100% PC. Neither the grade nor the fineness of the slag was specified. Mortar bars were prepared and tested according to ASTM-C1260 for 14 day expansions. In the ASTM-C1260 test, an abundant supply of alkalis is available from the solution in which specimens are submerged. Concrete prisms were also fabricated and tested according to ASTM-C1293 for 2 year expansions. In the ASTM-C1293 test, only the alkalis from the cement paste are available for the duration of the test

(i.e. not submerged in alkali rich solution). The cements with alkali contents of 0.46 and 0.98% were used for the C1293 and C1260 tests respectively. The results are shown in Table 2.5.

Table 2.5 Results of ASR Expansion Tests on GGBFS Specimens Tested According to ASTM-C1260 and ASTM-C1293, (Detwiler, 2003).

ASTM-C1260	Expansion at 14-days, %	ASTM-C1293	Expansion At 2-years, %
Mixture ID		Mixture ID	
<i>Failure Criteria</i>	<i>> 0.10 %</i>	<i>Failure Criteria</i>	<i>> 0.04 %</i>
Control (0.98)	0.25	Control (0.46)	0.14
35% GGBFS	0.17	35% GGBFS	0.06
50% GGBFS	0.06	50% GGBFS	0.03
15% Low-CaO F.A.	0.09	15% Low-CaO F.A.	0.06
25% Low-CaO F.A.	0.03	25% Low-CaO F.A.	0.04
15% Med-CaO F.A.	0.17	15% Med-CaO F.A.	0.12
25% Med-CaO F.A.	0.16	25% Med-CaO F.A.	0.07
15% High-CaO F.A.	0.20	15% High-CaO F.A.	0.20
25% High-CaO F.A.	0.18	25% High-CaO F.A.	0.15

As can be seen by observing the results, GGBFS replacements of 35% decreased the expansions during both procedures; however it was not enough to achieve passing scores. For both the tests performed, the 35% GGBFS specimen did not meet the passing criteria. However, only two specimens passed the ASTM-C1260

which included the 50% GGBFS specimen. The 50% GGBFS specimen outperformed all specimens for the ASTM-C1293 test with an expansion of only 0.03%. Although not achieving a passing score, it should be noted that the specimen with only 35% GGBFS performed better than all specimens except the 25% Low-CaO F.A. during the ASTM_C1293 test.

Higgins and McLellan (2009) performed extensive studies over a period of ten years to study the effectiveness of GGBFS in reducing ASR expansion. The concrete mixtures were made such that the total reactive alkali contents varied from 0.31, 0.37, 0.44 and 0.50 lb/ft³ (4.96, 5.92 and 7.05 kg/m³) using three types of cement and two types of GGBFS. Several hundred concrete prisms were made with varying amounts of GGBFS from 0-70% replacements. Natural aggregates were used that were known to have caused ASR expansion in structures. The cement types used were designated low, medium and high for alkali contents of 1.15, 0.87 and 0.54% respectively. The testing was performed in the UK and methods as well as standards varied from ASTM Standards and Methods. Whether or not the UK has a standard method of rating GGBFS is unknown, however neither the grade nor the fineness of the slag used was specified. The alkali contents for the two types of slag designated low and high were 0.58 and 0.83% respectively. Two different temperatures were used during testing, 68 °F and 100 °F (20 °C and 37.8 °C). The higher temperature was chosen to investigate whether accelerated expansions would take place and more importantly, whether the results from both tests would correlate. The specimens were tested for a total of ten years according to British Standard Institution Procedure DD 218:1995.

The results of this study showed that the accelerated curing temperatures did increase the expansion rates by as much as 3.0 %. The results for the accelerated tests also correlated well with those from the standard test method. The author found that in general as the amount of alkali increased, the expansions increased and as GGBFS

content increased, the expansions decreased. What is also interesting is that none of the specimens made with 70% GGBFS expanded significantly, even with total alkali contents up to 0.6 lb/ft^3 . For the specimens made with 50% GGBFS, expansions greater than .10% were only examined when the total alkali contents were far in excess of what would be found in the field. They also determined that half of the alkali content is contributed by the slag itself only when used in replacement percentages up to 25%, and beyond this the alkali contents of the slag no longer become a factor.

2.2.5 Summary

This literature review showed that there can be both positive and negative effects from the use of GGBFS in a concrete mixture. There was evidence that higher dosages of air-entraining admixture and HRWRA may be needed if GGBFS is used. This dosage increase appears to be related to the fineness of the GGBFS and with increasing fineness, one can expect to see an increasing demand on admixture dosage. Temperatures are expected to decrease in a concrete mixture and the degree will depend on the amount of GGBFS used as replacement. The workability of concretes made with GGBFS is expected to increase with increasing slag content. This increase in workability will also be related to the grade and fineness of slag used and will decrease as these attributes increase respectively. Concretes made with GGBFS will typically show higher later age strengths than earlier age when compared with control mixtures made of 100% PC. The higher grading of slag used will tend to reflect higher strengths and strength gains due to reactivity. The optimum replacement levels for the greatest strengths tend to be around 50% for GGBFS. Typically, strengths have shown an increase up to 50% and decreasing or similar strengths thereafter when compared to control mixtures.

The permeability of concrete will decrease with increasing slag content. This decrease in permeability is associated with several other increases in durability for GGBFS concretes. The use of GGBFS will typically increase sulfate resistance and may or may not have an effect on the freeze-thaw durability. The increase in sulfate resistance will increase with increasing slag content but may exhibit conflicting results in actual field conditions where sulfate rich soils are encountered. The resistance to freeze-thaw will typically decrease with replacement levels in excess of 50%. The resistance to ASR expansion will most probably increase with increasing slag content when compared with concretes made of 100% PC and similar aggregates. Significant increases in resistance to ASR expansion have been shown at typically higher replacement levels of 50% or more.

2.3 Waste Glass as Aggregate

The majority of crushed glass (approximately 80%) that exists in the recycling stream comes from bottles. These bottles will primarily be from beer and soda but may also come from other food sources. The recycling of glass is difficult and costly due to the usual need of separating the glass colors before recycling. This will depend on end use though as the glass will maintain its color after recycling. Not all glass can be recycled due to impurities and contaminants which are difficult to remove and because frequently the bottles are broken or damaged which makes re-use impossible. Difficulties in achieving the cullet (crushed waste glass) specifications required by many bottle manufacturers have resulted in a low market value for crushed glass and the bulk of crushed glass ends up in landfills. In 2007 an approximated 13.6 million tons of waste glass was generated and only 24% of this was recycled (Verdugo, 2009). The excessive shipping costs for bulk glass to manufacturing plants are also prohibitive due to the high density of glass.

The use of glass in concrete is gaining interest because of the potential bulk use as a fine aggregate. However, many earlier studies on the use of glass in concrete have shown that poor quality concrete is a result due to alkali-silica reactivity between the glass particles and the cement paste. This trend is changing as more ways to mitigate the ASR potential are becoming known. Due to the taboo of using glass in concrete, the research is limited and a lot of information on the subject aside from ASR studies is minimal. Some newer research into glass as aggregate has found that pulverized glass (powdered glass) will actually mitigate potential ASR and may also act as a pozzolanic material.

2.3.1 Production

Waste glass comes from glass recycling plants which collect the glass from households and commercial facilities. The glass is stockpiled at the recycling facility and may or may not be separated based on color. The separation of glass with regards to color is done because glass will maintain its color after recycling and depending on end use this separation may be required. The color of the glass will also dictate the chemical composition because of the chemicals added during original production to alter the color. This separation of glass at the recycling plant may also be required because of this chemical incompatibility of different colors. The primary colors that make up the majority of the glass in the recycling mainstream are brown (amber), green and clear (transparent).

The typical process of recycling bottles involves multiple crushers which decrease the size of the glass particles more after each crusher. The glass moves along a conveyor belt from one crusher to the next while vacuums and magnets remove any labels or other foreign objects. After all crushing has taken place the glass drops into a hopper and is ready for the furnace where it will be melted down. At this stage the

crushed glass is called “cullet”. Washing of the cullet may or may not take place prior to entering the furnace and depends on the plant operation.

2.3.2 Physical and Chemical Properties

Since most of the glass that makes up the recycling stream is composed of containers such as beer and other carbonated beverages, the chemical and physical composition of these glasses will be discussed. Glass is an amorphous (non-crystalline) solid structured material. The type of glass used to make common beverage containers is called soda-lime glass and is primarily made from quartz with the addition of other materials to help the fusing process. The term soda-lime comes from the addition of lime (CaO) during the manufacturing process. Soda lime glass has a smooth surface and is non-porous in nature (non-water soluble). This smooth surface raises concern with regards to strength since the weakest link in the concrete matrix is the bond between aggregate and cementitious paste. Replacing a natural sand aggregate which has a rough surface with a smooth surfaced glass may result in decreased strength at the interfacial transition zone (ITZ).

Ordinary soda-lime glass will be colorless without the addition of chemical compounds to change the tint. FeO and Cr_2O_3 are added to the glass to make the green colored bottles. Sulfur, carbon and iron salts are added to the glass if amber or brown colored bottles are made (www.glassproperties.com, 2011). As will be discussed later, the color of the glass is believed to have significant impacts on the ASR potential due to the variation in chemical composition. Table 2.6 shows the typical molecular compounds and percentages that make up soda-lime glass.

Table 2.6 Typical Chemical Compositions Percentages of Soda-Lime Container Glass, (Weihua Jin, 2000)

Compound	Clear Glass (%)	Amber Glass (%)	Green Glass (%)
SiO ₂	73.2 – 73.5	71.9 – 72.4	71.27
Al ₂ O ₃	1.7 – 1.9	1.7 – 1.8	2.22
CaO + MgO	10.7 – 10.8	11.6	12.17
Na ₂ O + K ₂ O	13.6 – 14.1	15.8 – 14.4	13.06
SO ₃	0.20 – 0.24	0.12 – 0.14	0.052
Fe ₂ O ₃	0.04 – 0.05	0.30	0.599
Cr ₂ O ₃	-----	0.01	0.43

Aside from the properties associated with the glass manufacturing itself, recycled waste glass will also contain other impurities on the surface that are associated with the contents the glass container once held. Sugars and other organic contaminants may be present. Washing of the waste glass will typically remove most of these contaminants but this process can be costly. Other contaminants such as remnants of the paper labels and various metal objects (i.e. aluminum, bottle caps, etc.) not picked up by the magnets and vacuums during the crushing process can also be found. If aluminum remaining from labels or other sources is not removed the reaction between the alkalis and the aluminum will produce hydrogen bubbles. For this research project the glass was used “as received” from the recycling plant (unwashed).

The average specific gravity for soda-lime glass is 2.52, (en.wikipedia.org, 2011). The absorption capacity of the glass itself should be relatively 0.00%. However, due to the contaminants within the waste glass stream, especially small paper label fragments, will actually increase the absorption capacity slightly from

zero (C. Meyer, 2001). The fineness modulus, particle size and gradations of waste glass will ultimately depend on the recycling plants crushing technique and will vary.

2.3.3 The Effects of Waste Glass on Fresh Concrete Properties

2.3.3.1 Slump

Polley (1996) performed extensive studies to determine the effects of waste glass on fresh properties of concretes made with varying amounts of waste glass as aggregate. The study included various replacement amounts of both coarse and fine aggregate with waste glass. Waste glass replacement levels ranged from 0 (control) to 90% of total aggregate volume. They investigated the required amount of mixing water necessary to achieve a target slump of 50 mm (2.0 inch). Therefore the w/c was varied throughout this research project. These concrete mixtures also had varying amounts of Class F fly ash used as supplementary cementitious material ranging from 20 to 30%.

The results showed a linear trend of increasing water content as total glass content increased. What was also observed and noted in the report was the segregation of the waste glass with the cement paste. This was most notable with the coarse glass aggregate. A poor bond between the glass and cement paste existed and the angularity of the glass also contributed to the poor workability. The concretes were difficult to work with and finish properly (actual sidewalk slabs were placed for this research). They also core drilled these slabs at a later date and noted that the consolidation at the bottoms was typically poor.

Ahmad Shayan and Aimin Xu (2005) examined the fresh concrete properties of several concrete mixtures composed of varying amounts of waste glass as fine aggregate. Replacement levels of 40, 50 and 75% were investigated for this research.

A constant w/c value as well as total cementitious content was used. Glass powder was used as a supplementary cementitious material on several mixtures. Table 2.7 below summarizes the mixture properties and fresh properties for several selected mixtures. The CGS in the mixture identification signifies crushed glass sand, or waste glass used as fine aggregate and the replacement level used. It was indicated that HRWRA was used on some mixtures but the documentation was vague regarding which mixtures, as well as dosage rate.

Table 2.7 Mixture Properties and Slump Results of Concretes Containing Waste Glass as Fine Aggregate, (Shayan et al, 2005).

Mixture ID		Cement (lb/yd ³)	Glass Powder (lb/yd ³)	w/c	Slump (inch)
#1	Control	640	0	0.49	2.75
#7	40% CGS	448	192	0.49	2.00
#6	50% CGS-1	512	128	0.49	3.15
#9	50% CGS-2	640	0	0.49	2.75
#8	75% CGS	448	192	0.49	2.00

These results are interesting because it does not appear that the glass had any effect on the workability of these mixtures. For instance, when comparing Mixture #1 (control) with Mixture #9 the slumps are identical. Both mixtures had the same cementitious and only varied in waste glass replacement which was 50% for mixture #9. The same similarities can be observed when comparing Mixture #7 with Mixture #8. The replacement of fine aggregate with waste glass between these mixtures is

35%, but the slumps were identical. A relationship can be seen between glass powder and increase in slump when observing Mixture #6 and Mixture #9. As mentioned previously, the usage of HRWRA was specified but the information was vague and it is assumed that all mixtures contained some amount of HRWRA. The author did point out that the workability of the waste glass mixtures was harsh and that the harshness increased as glass content increased.

2.3.3.2 Air Content

The air contents of concretes made with crushed glass as aggregate do not show any significant differences when compared with natural aggregate concretes based on the studies investigated. Polley (1996) investigated multiple replacement levels of natural coarse and fine aggregates with crushed waste glass and found only a small difference in AEA dosage requirements. However, the small additional AEA dosage that was required on mixtures that had replacements of aggregate with waste glass also contained fly ash and the increase was associated mainly with the fly ash, not the waste glass. There was a difference in required AEA dosage for the glass powder mixtures once the replacement level exceeded 15%. However, powdered glass is not considered an aggregate as the fineness can be well in excess of even cement ($8000 \text{ cm}^2/\text{g}$ ($3906 \text{ ft}^2/\text{lb}$)).

2.3.4 The Effects of Waste Glass on Hardened Concrete Properties

2.3.4.1 Strength

Polley (1996) performed extensive studies to determine the effects of waste glass on the strength of concretes made with varying amounts of waste glass as aggregate. The glass used was a combination of various colors directly from the recycling plant. The study included various replacement amounts of both coarse and fine aggregate with waste glass. The w/c values ranged considerably throughout this study due to the increased water demand as glass content increased. A target slump of 50 mm (2.0 inch) was specified and the additional water added to achieve this workability sometimes caused a useless mixture. These concrete mixtures also had varying amounts of Class F fly ash used as supplementary cementitious material ranging from 20 to 30%. A low alkali cement and a moderate alkali cement were used.

The results of these studies are too extensive to tabulate and therefore only the key findings will be discussed. The results of this study showed that as glass content increased the strengths decreased. This relationship was most pronounced with the coarse glass aggregate concretes and was attributed to the plane of weakness associated with the larger glass particles. The glass particles are also not as strong as natural aggregates and are susceptible to fracture which may cause lower strengths as well. The effects of reduced strength decreased as the fineness of the glass increased. The low alkali cement mixtures exhibited greater strengths than the moderate alkali mixtures which suggest that ASR was a contributing factor to strengths. A reduction in strength loss from 20% to only 5.0% when using low alkali cement as opposed to moderate alkali cement was realized. They concluded that an optimum replacement of 20 to 24% and limiting the glass to only fine aggregates would limit the decrease in strengths to only 5.0% when compared with a similar control mixture. Another substantial observation made was the differences in strength when comparing the

unwashed glass to the washed glass concrete mixtures. They investigated similar mixtures that used the same replacement amounts of glass and either used the glass “as received” or washed the glass prior to usage. For mixtures containing 40% coarse glass aggregates the difference in strengths is 20%. For mixtures containing 25% glass as fine aggregate the difference in strengths are 40% at 28-days of age and 44% at 56-days of age. This reduction in strength was attributed to the presence of sugars and other contaminants remaining on the glass.

M. Mageswari et al (2010) investigated the strengths of concretes containing various replacement amounts of natural sand fine aggregate with waste glass fine aggregate. This waste glass was composed entirely of sheet glass scraps collected from local shops near the area where the research was conducted. The sheet glass was crushed and graded similar to the fine aggregate it was to replace. The glass also consisted of multiple colors and was not separated prior to usage. To investigate the effects on concrete strength, replacement levels of 10, 20...and up to 100% of the fine aggregate was used. Their results indicated that the optimum replacement levels were between 10 to 20% of the fine aggregate. All mixtures showed an increase in strength when compared to the control mixture for the earlier age groups (up to 90-days). However, at 180 days of age all mixtures showed decreased strengths when compared to the control specimens. Although the concretes were not tested for this reaction, the strength reductions were attributed to ASR.

Ahmad Shayan and Aimin Xu (2005) examined the effects of glass aggregate on concrete strengths for various replacement ranges of fine aggregate. They also investigated the effects of glass powder (fineness of $8000 \text{ cm}^2/\text{g}$ ($3906 \text{ ft}^2/\text{lb}$)) on the strengths of concrete. Both the powdered glass and glass fine aggregate came from soda-lime waste glass. The mixtures were designed and later compared to a 5800 psi (40 MPa) 28-day structural concrete mixture. Specimens were tested at 7, 28, 90 and 404-days of age. Table 2.8 shows several key mixture designs as well as 28 and 404-

day strengths. The CGS in the mixture identification refers to crushed glass sand as replacement of natural sand. Some mixtures had glass powder as a supplementary cementitious as well. It should be noted that the compressive strengths were approximated from bar chart as no tabulation of values was provided. Therefore, minor discrepancies in data may exist.

Table 2.8 Mixture Properties and Compressive Strengths of Concretes Containing Waste Glass as Fine Aggregate, (Shayan et al, 2005).

Mixture ID		Cement (lb/yd ³)	Glass Powder (lb/yd ³)	w/c	Compressive Strength (lb/in ²)	
					28-Day	404-Day
#1	Control	640	0	0.49	8412	11,168
#7	40% CGS	448	192	0.49	5076	8557
#6	50% CGS-1	512	128	0.49	5221	8412
#9	50% CGS-2	640	0	0.49	6237	8992
#8	75% CGS	448	192	0.49	4641	8267

When observing the compressive strengths of the waste glass concretes, only Mixture #9 achieved the required strength of 5800 psi (40 MPa) at 28-days of age. All mixtures achieved the required strength by 404 days. The slowest strength gain came from the 75% CGS mixture but this mixture also had a 30% cementitious replacement with glass powder. Although all mixtures achieved significant results, the decrease in strength due to waste glass is evident when compared with the control mixture. When comparing the control mixture (Mixture #1) with Mixture #9 which both had 100%

PC as cementitious, the reduction in strength is over 2000 psi (13.8 MPa) for both age groups. The inclusion of the glass powder also had significant effects on strengths, but the reduction in PC used may well be worth this reduction. Unfortunately information regarding the waste glass was vague at best. No information was given as to whether or not the glass was processed further before mixing or whether or not any washing took place.

2.3.4.2 Permeability

Ahmad Shayan and Aimin Xu (2005) investigated the effects of crushed glass aggregate and powdered glass cementitious replacements on the permeability of hardened concrete at 380-days of age. The testing was performed according to ASTM C 1202 procedures. The replacements of fine aggregate with waste glass were 40, 50 and 75% of the total fine aggregate content. Some mixtures also incorporated powdered glass as supplementary cementitious. The w/c was held constant as well as the total cementitious content. This study involved actual field testing in which slabs were placed and finished outside at a test facility. Cylinders were cored from the slab at the specified time of testing. When the slabs were constructed separate cylinders were also cast from the same batch and cured in the laboratory for comparison with field specimens. Table 2.9 summarizes the results from several selected tests of both the lab cured specimens as well as the field cured specimens for comparison. These results were scaled from bar graphs and therefore small discrepancies may exist.

Table 2.9 Mixture Properties and RCPT Result of Concretes Containing Waste Glass as Fine Aggregate at 380 Days of Age, (Shayan et al, 2005).

Mixture ID		Glass Powder (lb/yd ³)	Field Charge Passed (Coulombs)	Permeability Class	Laboratory Charge Passed (Coulombs)	Permeability Class
#1	Control	0	2400	Moderate	2450	Moderate
#7	40% CGS	192	1200	Low	800	Very Low
#6	50% CGS-1	128	850	Very Low	1000	Very Low
#9	50% CGS-2	0	3400	Moderate	2600	Moderate
#8	75% CGS	192	650	Very Low	600	Very Low

The results show an increase in permeability as waste glass increases. When comparing field specimens for Mixture #1 with Mixture #9 the difference is 1000 Coulombs. Mixture #9 had 50% waste glass as fine aggregate and did not have powdered glass as cementitious. The laboratory specimens for these two mixtures achieved conflicting results for Mixture #6 with a difference of 800 Coulombs. The exact opposite of this trend was observed for mixtures #7 and #8. Mixture #7 had 40% glass content while Mixture #8 had 75% glass as aggregate. Mixture #7 with the lower amount of glass aggregate had significantly higher laboratory results when compared with Mixture #8. However, the field cured specimens both achieved similar results and a decrease in results was observed. Both of these mixtures as well as Mixture #6 did have powdered glass as cementitious added and this may have been the cause of the lower permeability's. When comparing Mixture #6 and Mixture #9 the effects of powdered glass on permeability can readily be seen.

2.3.4.3 Freeze-Thaw Durability

Polley (1996) investigated several different glass aggregate concrete mixtures for free-thaw durability. Most studies on glass aggregate are related to strength and ASR and therefore this was the only study that could be found on this subject. Several beams were fabricated with different glass contents and tested for 600 cycles of freezing and thawing. For this experiment, several mixtures with coarse aggregate replacements and fine aggregate replacements were investigated. The replacement amounts ranged from 12% to 36% for the coarse plus fine and 20% to 24% for the fine only. The degradation in durability factors for the coarse glass aggregate beams is more rapid than the fine glass aggregate beams. There is a correlation between increasing degradation rate and increasing glass content for the coarse plus fine glass replacement beams. This correlation does not exist for the fine glass only beams. All beams that had only fine glass replacements had durability factors greater than 85%. The coarse plus fine glass beams did not favor as well and several beams fell below 80%. The author pointed out that all beams that failed exhibited strong performance up to about 70 cycles before an alarming drop in durability factor occurred. Another interesting point made is that all beams (even the control) showed a decrease in durability factor for the first 10 cycles by around 6 to 7%. The mass loss observed for the coarse plus fine glass mixtures showed very similar results between 2.5 and 3.5% loss at 350 cycles.

The mass loss for the fine glass replacement beams showed more variations and were not as tightly grouped. These specimens performed worse than the combination specimens with mass losses between 2.5 and 6.5%. The severity of the mass loss for both the fine and fine plus coarse glass replacement specimens was also correlated with the amount of replacements. As glass content increased the mass loss increased.

2.3.4.4 Alkali-Silica Reactivity (ASR)

The alkali-silica reactivity between glass aggregate and cement is well documented. Many studies have shown that when crushed glass is used as an aggregate replacement, ASR is almost certain. However, more recent studies have also shown there are ways to successfully use glass in a concrete if proper techniques are employed. Mitigation techniques such as the use of pozzolans and even the type or color of glass has proven to be successful in reducing expansions caused by ASR to negligible levels.

In general, the causes of alkali-silica reactions are due to the alkali content in cement and the silica in the aggregate. The reaction between crushed glass and the alkalis from the cement can cause an expansive gel to form around the glass particle. This reaction can cause excessive internal forces and may ultimately destroy a concrete. ASR is a complicated reaction which involves many different factors. The general factors that can be used to control the effects of ASR in a concrete are: control of the alkali concentrations (primarily from cement), control of the amount of reactive silica (aggregate), control moisture penetration (permeability), control the pH in the pore solution and alteration of the alkali-silica gel that forms during the reaction (delay deterioration), (Mindess, 2003).

Most studies found that tested the potential for ASR in glass aggregate concretes followed the ASTM C 1260 procedures. This procedure is convenient because it lasts for only 2 weeks. However, this test as opposed to ASTM C 227, involves submergence of the specimens in a highly alkaline NaOH solution at elevated temperatures and is not representative of actual field conditions. This test has been shown to be highly reliable however and is the most common used (Weihua Jin, 2000). Two reports on ASR testing conducted with concretes containing glass will be discussed later. Most research on glass aggregates and potential ASR have come to

the following similar conclusions. Where reactive glass was used, the particle size appears to have a direct influence on the ASR. This is because the ASR process involves the reaction between a solid (aggregate) and a solution (cement paste) and the surface area of the aggregate will play a major role in reaction rate. There is also a pessimum size that will cause the most significant expansions. Most past studies that found this pessimum effect theorized that it was related to the variation in alkali to silica ratio. They argued that the pessimum behavior was associated with a pessimum gel ratio. However, these tests were based on ASTM C 227 which has a fixed supply of alkalis during the procedure.

There are two processes involved with ASR, gel formation and gel permeation. These two processes are both related to aggregate size. Gel formation will build up internal stresses within the concrete matrix, while gel permeation will tend to relieve these pressures. This is somewhat similar to the freeze-thaw process. Ice crystal formation causes excessive stresses in confinement, but where air voids are present the ice crystals will permeate to these areas and relieve stress. Gel formation predominates the coarser aggregate domain while gel permeation predominates the finer domain. This pessimum point is when there exists a balance between these two and the maximum expansion will take place. It has also been shown that decreasing the size of the glass particles to below a standard U.S. #50 sieve (i.e. passing) will eliminate most ASR potential.

Weihua Jin et al (2000) performed various studies into the effects of ASR on glass aggregate concretes. They first studied the effects of particle size on reactivity. They used a clear soda-lime glass for this portion of study and separated the glass into representative sizes corresponding to U.S. sieve sizes. They replaced 10% of the natural sand in the concrete with only one size for each mixture. Mortar bars tested according to ASTM C1260 indicated that the maximum, or pessimum size was the #16 sieve size (1.18 to 2.36 mm). What was also interesting is that for mortar bars

made with size #50 glass the expansions were identical to the reference bars made from only cement and natural aggregates. Mixtures made with glass particles equal to and smaller than #100 sieve actually showed less expansions than reference bars.

The second phase of testing involved the study of glass content on expansion rates. They made various mixtures with glass contents ranging from 0 to 100% of the fine aggregate. These mixtures were made with glass matching the gradations of the control specimens. A clear correlation was made between expansion rates and glass content. As glass content increased, so did the expansion rates. The relationship was rather linear as well.

The third phase of this study involved the investigation of glass color on ASR potential. Three separate groups of mortar bars were prepared for this investigation. Each group consisted of 10% replacement amounts of only one color of glass. The three groups consisted of green, brown (amber) and clear glass (previously investigated during phase I). The results showed that clear glass has the greatest expansions. Amber glass was shown to be considerably less reactive followed by green glass which showed non-reactive behavior and when compared with reference bars, showed less expansions. The pessimum size for the amber glass was found to be #8 size and although not reactive comparatively with the reference bars, #16 for the green glass.

They concluded that glass color and particle size has an effect on reactivity. They confirmed that a pessimum size exists and that this size depends on color and glass type but will shift toward smaller particles the more reactive it is. They concluded that green glass caused no expansions to speak of and that finely ground green glass can be a relatively low cost alternative to ASR mitigation in concretes containing reactive aggregates. They indicated this was caused by the Cr_2O_3 content in the green glass.

Amhad Shayan (2002) studied the effects of crushed glass and powdered glass (8000 cm²/g (3906 ft²/lb)) on the ASR potential of concretes made with crushed glass as fine and coarse aggregate replacements. The coarse aggregate particle size range was 12 mm to 4.75 mm (1/2 inch to 3/16 inch). The fine aggregate particle size range was 4.75 mm to 0.15 mm (1/2 inch to 1/200 inch). The waste glass used was a mixed colored soda-lime glass cullet from a local bottle recycling plant. The powdered glass was made from the cullet by further grinding. The Blaine fineness of the powdered glass was 8000 cm²/g (3906 ft²/lb). The testing procedure followed was ASTM C 1260 for 28-days.

They found similar results as Jin with regards to glass content and expansion. As the percentage of glass content increased the expansions increased linearly. They also found that replacements up to 30% of total aggregate were non-detrimental and expansions were insignificant when combined with low alkali cement. When investigating the gradation effects on expansion, they found direct correlations between particle size and expansion. The results show that expansions are negligible for particle sizes below 0.30 mm and that the coarse aggregates (greater than 0.60 mm) showed the greatest expansion rates. Furthermore, it was concluded that as the particle size decreases the ASR potential not only decreases but the fine glass particles will actually help suppress ASR when combined with the larger coarse glass particles. Powdered glass was also found to be a good suppressant of ASR expansions when used at cement replacement amounts of 20 to 30%.

2.3.5 Summary

Most studies on the effects of waste glass on concrete properties were limited to ASR research. This topic is the most widely investigated because of the almost certain problem that arises when using a known highly reactive aggregate. The use of waste

glass as aggregate is becoming more accepted if techniques are employed to suppress the ASR. It was shown that particle size does have a pronounced effect as well as glass color on the ASR expansion rate. However, most waste glass is not sorted and the use of only green glass (which is non-reactive) is not a practical approach. Reducing the size of the waste glass to below a #50 sieve appears to eliminate the ASR potential, but at the same time is no longer an aggregate but more of a cementitious material or powder. Incorporating glass fines does tend to decrease the ASR potential when used in combination with fine aggregate waste glass. The existence of a pessimum size of glass particles appears to be well founded and hovers around the #16 sieve size. The pessimum effect does not appear to be present when relating to content. As glass replacements increased the potential for ASR expansion also increased when mixed colored glass was used.

The absorption of the glass will be relatively zero, but the effects on workability will increase as content increases. Typically most mixtures made with glass as aggregate exhibited harsh characteristics and these increased as content increased. The effects of AEA will probably remain the same or slightly increase for concretes made with waste glass aggregate. The strength of the concrete will probably decrease with increasing waste glass content. This is due to the relatively low strength of glass when compared to natural sand aggregate. The bond between the glass particle and cement paste will not be as strong because of the glassy smooth structure. Whether or not the glass has been washed will also have major effects on concretes strength. A reduction in strength of up to 40% was observed when comparing washed to unwashed glass.

The permeability and freeze-thaw durability of concretes made with glass is not well understood or studied. Few studies could be found on these subjects. Of those found the general results showed that freeze-thaw durability's may decrease with increasing content. This decrease will also be more pronounced with increasing

glass size (i.e. coarse as opposed to fine). Once again, this can probably be related to the decreased strength and bond between glass particles and cement paste. The permeability of concretes using waste glass as aggregate will probably increase as replacement increases. This increase will depend on replacement amount and also on the fineness of the glass. The use of powdered glass actually decreases the permeability of concrete but this is more of a cementitious material and not an aggregate.

2.4 Recycled Concrete as Aggregate (RCA)

2.4.1 Production

The recycled concrete used as aggregates in concrete comes from buildings, roads, bridges and other civil structures that have been demolished. Only non-contaminated concrete is accepted at plants and cannot contain any trash, wood or other materials. In the past, most recycling plants would not accept concrete with reinforcing however nowadays these concretes are being accepted more and more. Any concrete that is suspected of having alkali-silica reactivity or other deleterious mechanisms will also not be accepted.

Typically the concrete is crushed to a reasonable size at the construction or demolition site before being transported to the recycling plant. There are portable crushers that can crush the concrete down into gravel at location and these are becoming more common as opposed to hauling large chunks to the recycling plant. At the recycling plant the concrete will be crushed further by primary and secondary crushers and filtered to remove contaminants and reinforcing steel. The reinforcing is removed by large magnets. The crushed concrete is then graded and washed. Most recycling plants will separate the crushed concrete into different piles which

correspond to a nominal maximum aggregate size (NMAS). Any contaminants removed including steel from reinforcing will then be sent to other recycling plants or landfills.

RCA usage has increased over the years due to increased savings in mixture design as well as an increase in studies and data supporting the use of RCA as a respectable aggregate. Most commonly RCA usage is primarily in roadways but usage in buildings is growing (PCA, 2002). Another new approach to recycling concrete and reusing as aggregate will also be examined. This newer approach involves reusing the concrete that gets returned to batch plants after a job has finished. This concrete is crushed and stockpiled similar to RCA but is called crushed concrete aggregate (CCA). The differences between these two aggregates are greater control of properties due to the known properties of the source concrete, and also in the lack of potential contaminants in CCA. CCA has never been used in the field and chemical contamination from chlorides will also not be a concern.

2.4.2 Physical and Chemical Properties

The physical and chemical properties of recycled concrete aggregate (RCA) will vary greatly depending on the source of the demolished concrete. These properties will be directly related to the original aggregate used and also on the cementitious materials. Crushed concrete will not only contain the coarse aggregate but also chunks of mortar containing the fine aggregate and cementitious paste. This paste will also surround portions of the coarse aggregate in varying amounts. Usually the NMAS will be consistent with the recycling plants designation when tested separately. Even if the concrete being recycled does not show signs of distress from alkali-silica reactivity, it should be tested because the aggregate may have been reactive but low alkali cement or other measures may have been used. Chloride content should also be tested

because of the high volumes of recycled concrete that come from roadways and the chlorides used to melt snow may be present. ACI 555R (2001) has developed requirements pertaining to the amounts of deleterious materials acceptable in coarse and fine RCA depending on type of deleterious materials.

The unit weights and specific gravity will also vary greatly depending on the source but are typically lower than a normal virgin aggregate of similar properties. This is caused by the hydrated cement mortar still attached to the aggregates as the mortar will typically have lower density than the source aggregate (ACI 555R, 2001). The absorption capacity of recycled concrete aggregate will usually always be higher due to the increased porosity of the mortar chunks and cementitious paste surrounding the aggregate. Absorption values will typically range between 3 and 10% and will increase as NMAS decreases (PCA, 2002). The shape of RCA will typically be angular and resemble crushed rock. This will also cause workability concerns as opposed to round natural stone.

2.4.3 The Effects of RCA on Fresh Concrete Properties

2.4.3.1 Slump

Gholamreza Fathifazl et al (2009) examined the effects of RCA on the fresh properties of concrete. For this particular study a new mixture design was developed to incorporate the existing mortar content within the RCA. As previously stated, the RCA will contain chunks of mortar as well as cementitious paste surrounding the aggregates. This new method accounts for this preexisting mortar in the design process and subtracts this amount from the new cementitious quantities. Several mixtures were developed from varying RCA sources and compared with a similar control mixture composed of normal virgin aggregates and PC. A total of six RCA

mixtures were developed and tested. Five of these six mixtures contained varying amounts of water reducing admixtures. All RCA mixtures had significantly less slumps than the control mixture, even though the control mixture did not have HRWRA. This was attributed to the reduction in “new mortar” caused by this new design method.

The U.S. Department of Transportation, Federal Highway Administration (FHWA) performed an extensive survey of all state transportation agencies to investigate the usage of RCA in roadway construction (U.S. Department of Transportation, FHWA, 2004) They found that although 41 states currently recycle aggregate from demolished roadways, only 11 of these states reuse the RCA in new roadways. They recommend that RCA must be treated as a lightweight aggregate and RCA requires special attention in order to achieve the same workability as a normal PC concrete. This is due in part to the high absorption capacity and also because of the coarseness of the RCA. They estimate that concretes made with RCA will require on average 5% more water and if the percentage of fines increases as much as 15% more water may be needed. They also recommend that when RCA is used it must be in saturated surface dry (SSD) condition prior to batching and that RCA stockpiles be constantly wetted with water. They mentioned that several state agencies have reported decreased workability in concretes made with RCA but this was attributed to the lack of preparation and increased fines content. Most state agencies that use RCA have limited the amount of fines to 20% due to concerns with workability.

Mirjana Malešev et al (2010) investigated the fresh concrete properties of concrete made with RCA. The RCA replacement was investigated at 50 and 100% of the total coarse aggregate content. Natural sand was used for the fine aggregate on both these mixtures and for the 50% RCA mixture, the remaining 50% of coarse aggregate was a natural aggregate. A control mixture was also made with the same natural sand as the RCA mixtures and the same natural coarse aggregate as the 50%

RCA mixture. A Type-II PC was used for all mixtures and the w/c was started at 0.514 and varied depending on mixture.

It is unclear which design method was used when designing these mixtures. It is assumed these mixtures were designed based on SSD and the additional water is due to the moisture contents and absorption capacity of the RCA. Therefore it is assumed this extra water reported is actually required to achieve the same w/c when comparing the control mixture with the RCA mixtures. The average absorption capacity of the RCA aggregate was 4.0% and the percentage of fines was less than 1.0%. To study the effects on workability two methods were employed, the first was to test the concrete mixtures immediately after batching and the second involved testing slump after 30 minutes of mixing. This second method was employed because of a belief that the workability of the concrete should be tested at a time similar to what would be seen in the field. Table 2.10 summarizes the results of these three mixtures.

Table 2.10 Slump Test Results of RCA Concrete Mixtures (Malešev, 2010)

Concrete Mixture ID	W/C (ratio)	Immediate Slump, (cm)	30 min Slump, (cm)
Control (Natural Agg.)	0.514	16	10
50% RAC	0.568	14.5	8.5
100% RAC	0.620	11	9

The results indicated that immediately after batching the RAC mixtures showed decreased slump values, and these values decreased as RAC content increased. However, after 30 minutes of mixing the slumps for both RAC mixtures decreased but not as substantially as the control mixture. After 30 minutes of mixing the slumps of all three mixtures were quite similar. This indicates that mixing time is important and may need to be considered when using RCA in concrete. The cause of this was attributed to the additional mixing time and also to the additional water required by the RAC mixtures. However, as previously stated it is unclear whether or not this “extra” water was indeed extra or simply that which would be required based on conditions of the aggregate, (i.e. absorption capacity and moisture content).

2.4.3.2 Air Content

According to the U.S. Department of Transportation, Federal Highway Administration (2004), the use of air entraining admixtures in concrete made with RCA should follow the same guidelines as any other natural aggregate and testing should be conducted to determine the proper dosage. They also indicated that if the source concrete from which the RCA came from performed poorly in regards to durability (i.e. freeze-thaw resistance) then the RCA should not be used in new concrete that will be susceptible to freezing and thawing cycles. The source of RCA is sometimes difficult to determine and selecting an RCA from a known source may not be an option. They also indicated that AEA dosage in general may be slightly lower due to increased fines and the angularity of the RCA aggregate.

According to ACI 555R (2001) the air contents of concretes made with RCA will vary more than similar mixtures made with natural aggregates, but the general trend was that air contents would be slightly higher. Studies performed by Karthik Obla et al (2007) support these findings. They investigated the effects of crushed

concrete aggregate (CCA) on fresh and hardened concrete properties. They studied various concrete mixtures with different replacement levels of CCA ranging from 10 to 100% and found that the air contents of concretes made with CCA tended to be only slightly higher than the control mixture made with natural aggregate. This trend became more noticeable as the amount of CCA fines increased. The air contents measured were entrapped air only because AEA was not used for these mixtures.

2.4.4 The Effects of RCA on Hardened Concrete Properties

2.4.4.1 Strength

According to ACI 555R (2001), the strengths of concretes made with RCA will depend greatly on the source concrete. If concrete is made from varying sources of RCA then the strength trends will also vary. Based on previous studies of concrete containing RCA, if a single source for the RCA is used the variation in strength can be minimized. Table 2.11 summarizes results of compressive strengths for concretes with known sources and strengths with concretes made with RCA from the same sources. Although the w/c varied considerably in the source concretes, the new RCA concrete w/c was held constant.

When observing the Table 2.6, there doesn't appear to be a good correlation between source concrete strength and RCA concrete strength. There also is not a good correlation between source concrete w/c and the RCA concrete strength as well. The highest source w/c values do appear at the lower end of the strength spectrum, but this was not always the case. When comparing source strength to RCA strength, there is not much of a correlation at all. The highest strength RCA mixtures did come from the higher strength source concretes but this was not always the case as two of the highest strength source mixtures with accompanied low w/c values were also some of

the lowest RCA strength contributors. The only information provided for these mixtures was the w/c but cementitious content as well as other factors may have played a role in the scattering of data.

Table 2.11 Compressive Strengths of Source Concretes and Concretes Made With RCA From Same Source, (ACI 555R, 2001).

Source Concrete w/c	Source Concrete 15-year Compressive Strength, (lb/in²)	RCA Concrete w/c	RCA Concrete 28-Day Compressive Strength, (lb/in²)
0.53	10,900	0.57	7120
0.50	10,600	0.57	6870
0.59	9050	0.57	6280
0.65	8600	0.57	6250
0.65	9850	0.57	6040
0.67	7470	0.57	5840
0.50	8980	0.57	5770
0.80	5640	0.57	5510
0.50	12,300	0.57	5340
0.50	9290	0.57	5100
0.81	6100	0.57	4640
0.53	10,600	0.57	4380

Gholamreza Fathifazl et al (2009) examined the effects of RCA on the hardened properties of concrete. For this particular study a new mixture design was

developed to incorporate the existing mortar content within the RCA. This new method accounts for this preexisting mortar in the design process and subtracts this amount from the new cementitious quantities. This new method of design is called the Equivalent Mortar Volume method (EMV). Several mixtures were developed from varying RCA sources and compared with a similar control mixture composed of normal virgin aggregates and PC. A total of six RCA mixtures were developed and tested. To illustrate the need for a new mixture design when RCA is used, two methods of design were used when designing each RCA mixture. The new proposed design method was used on mixtures and designated with an "E-" at the beginning. The conventional ACI method of design was also used for comparison and designated with a "C-" at the beginning. Depending on what type of supplementary cementitious material was incorporated in these new concretes, if any, was also determined and designated as "B-" for GGBFS, "F-" for fly ash and "C-" for no supplementary used at the end of the mixture ID. For example, the mixture designated as "ERAC-B" indicates this mixture was designed with the new method and the cementitious contained GGBFS. The mixture "CRAC-C" was designed by the traditional ACI method and the cementitious content of the concrete only contained cement. The results of compressive testing performed on the RCA mixtures as well as the control mixture designated as NAC-C are shown in Figure 2.3. It should be noted that these values were scaled off a bar chart as no table reporting the exact values was available. Therefore, minor discrepancies may exist. It was also assumed these specimens were tested at 28 days of age as this information was also not given.

The results of the compressive tests show that this new method of design produces comparable strengths with the ACI method. It should be noted that on average the mixtures designed by this new method had 22% less new cement due to the existing mortar being accounted for in the mixture design. This could lead to substantial cost savings. What is also interesting is that GGBFS and Cement RAC

mixtures obtained greater strengths than the control mixture. The author attributes this greater strength to the stronger interfacial transition zone (ITZ) between the RCA and cementitious paste. It is believed this ITZ is stronger between RCA and paste as opposed to a natural aggregate and paste.

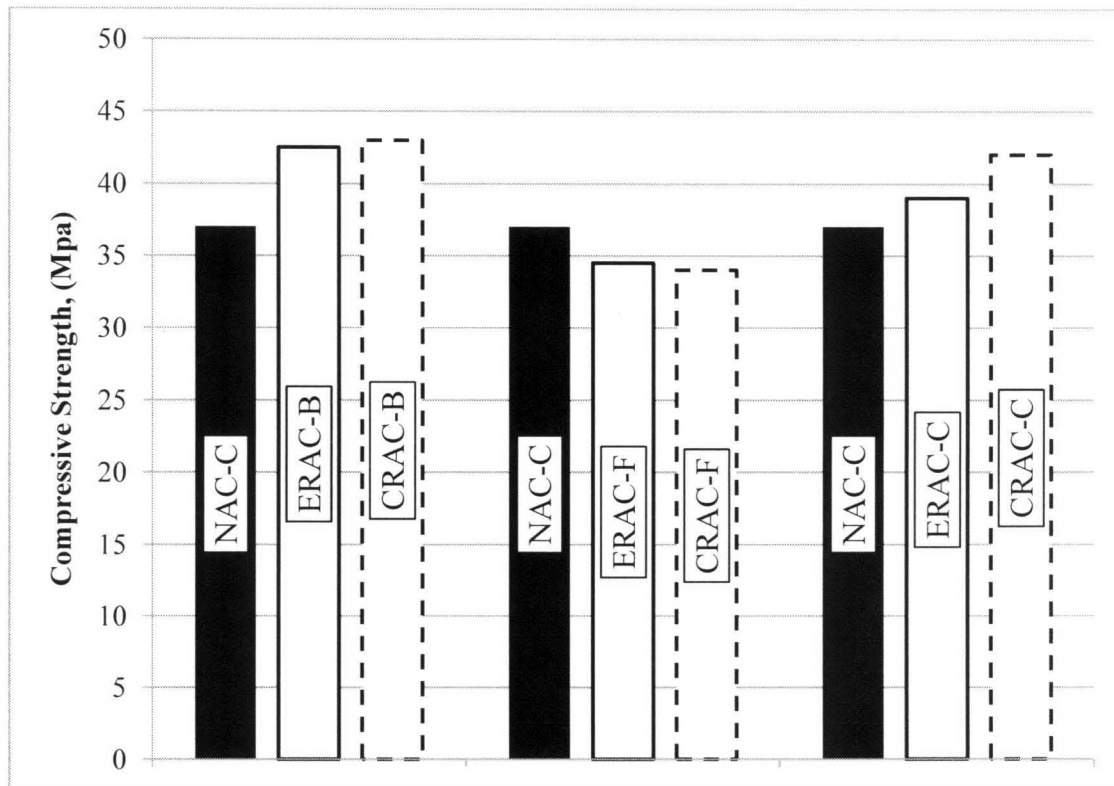


Figure 2.3 Compressive Strengths of RCA Concrete Mixtures Designed with the EMV Method and ACI Method, assumed 28-Day age, Fathifazl et al (2009).

Mirjana Malešev et al (2010) investigated the hardened concrete properties of concrete made with RCA. The RCA replacement was investigated at 50 and 100% of the total coarse aggregate content. Natural sand was used for the fine aggregate on

both these mixtures and for the 50% RCA mixture, the remaining 50% of coarse aggregate was a natural aggregate. A control mixture was also made with the same natural sand as the RCA mixtures and the same natural coarse aggregate as the 50% RCA mixture. A Type-II PC was used for all mixtures and the w/c was started at 0.514 and varied depending on mixture. The compressive strength was tested at 2, 7 and 28 days of age. Figure 2.4 shows the average results of these tests.

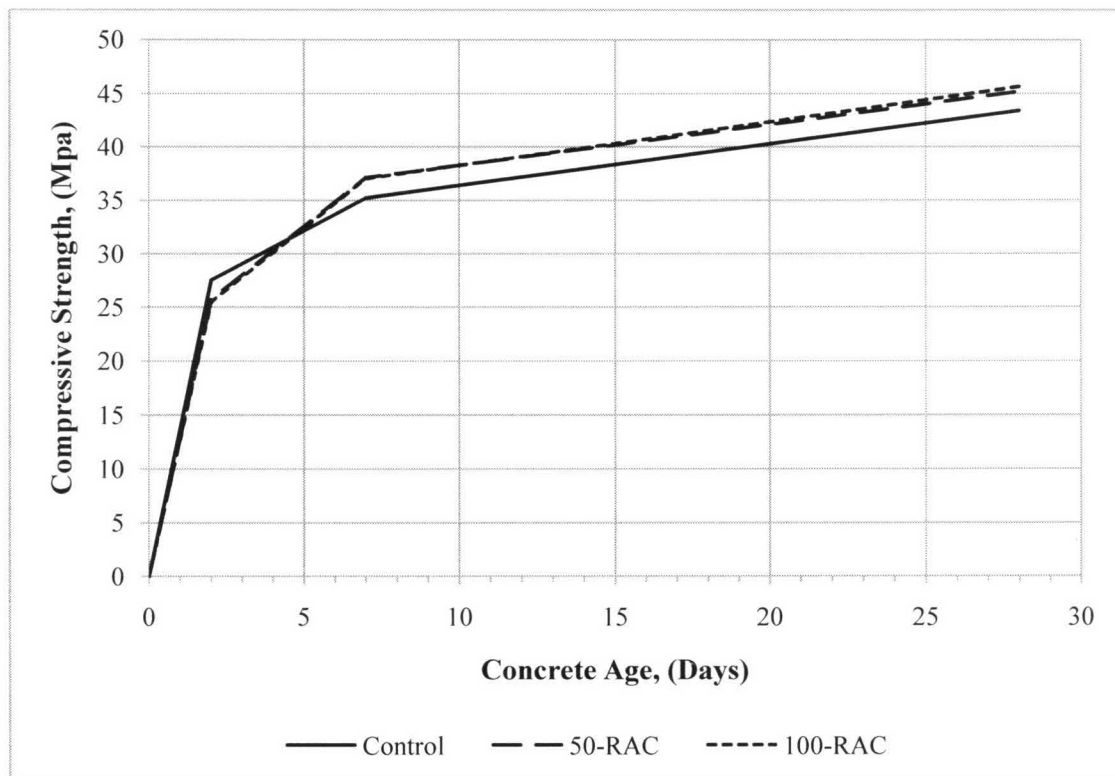


Figure 2.4 Average Compressive Strength of RCA Concrete and Control Concrete, Mirjana Malešev et al (2010).

The results of this study show that early age strengths for both the 50 and 100% RAC mixtures was slightly below that of the control mixtures made from 100% natural aggregates. After day 2 however the RAC mixtures gained strength more rapidly and overtook the control mixture for both the 7-day and 28-day strengths. This difference is small however and in comparison these mixtures were all quite comparable in strength. The w/c for the control mixture, 50-RAC and 100-RAC were 0.514, 0.568 and 0.620 respectively. Although strengths are comparable between these three mixtures, it was noted that the modulus of elasticity was lower for both of the RAC mixtures.

Karthik Obla et al (2007) performed extensive testing to determine the effects of CCA on concrete strength. They were interested in whether or not source concrete strength had an impact on new CCA concrete strength. Multiple concrete mixtures with different replacement levels of CCA ranging from 10 to 100% were developed and tested. The types of CCA used also varied and were categorized based on original design strength of the source concrete. Four types of CCA were used; 1000, 3000, 5000 psi (6.9, 20.7 and 34.5 MPa) and an “uncontrolled” combination. The 1000, 3000 and 5000 psi (6.9, 20.7 and 34.5 MPa) CCA came from known sources and was in essence “controlled” with regards to aggregate properties. The uncontrolled CCA was labeled as “pile 1” because it came from the general recycled concrete pile at the batch plant and is a general mixture of various recycled concrete.

Two different methods of incorporating the CCA into the mixture were used; one included the CCA in an “as received” state from the crusher and the other used pre-designated “Coarse” CCA aggregate. The “as received” CCA had a percentage of fine aggregate included within and therefore percentages of virgin fine aggregate were subtracted from the mixture design. If the “Coarse” method was used, only a percentage of the coarse aggregate was replaced and 100% virgin fine aggregates

were used. Table 2.12 summarizes the results from compressive strength tests performed at 7, 28 and 90-days of age.

Table 2.12 Compressive Strengths of Concretes Made with Varying Amounts of CCA, Karthik Obla et al (2007).

Mixture ID		Source Concrete Strength (lb/in ²)	w/c	7-Day Strength (lb/in ²)	28-Day Strength (lb/in ²)	90-Day Strength (lb/in ²)
1	Control-1	N/A	0.58	3080	4100	4740
2	Control-2	N/A	0.58	2980	3930	5350
3	10% T-CCA	1000	0.55	2910	3990	4670
4	20% T-CCA-1	1000	0.59	2410	3630	3790
5	20% T-CCA-2	3000	0.58	2800	3690	4450
6	20% T-CCA-3	3000	0.58	2610	3760	4570
7	20% T-CCA-4	3000	0.58	----	3900	4390
8	30% T-CCA	3000	0.59	2800	3890	4720
9	20% T-CCA-5	Pile 1	0.58	2590	3410	4530
10	50% C-CCA	1000	0.55	2640	3470	4330
11	100% C-CCA-1	1000	0.52	2460	3180	3630
12	100% C-CCA-2	3000	0.59	2730	3930	4270
13	100% C-CCA-3	3000	0.59	----	----	4220
14	100% C-CCA-4	5000	0.57	2740	3790	4810
15	100% C-CCA-5	Pile 1	0.68	2140	2690	3190
16	100% C-CCA-6	Pile 1	0.69	----	2840	3360

At 7 days of age all CCA concretes exhibited slightly reduced strengths when compared with the two control mixtures. When comparing the "coarse" mixtures (10-16) to the "as received" mixtures (3-9), the coarse mixtures tended to have slightly lower strengths on average. When comparing source concrete strengths for the "coarse" group there is a slight correlation between strength of new concrete versus source concrete strength. However, this cannot be said for the "as received" group of mixtures as the highest strength came from mixture 3. This may be due to the decreased percentage of CCA as well. This same trend can be noticed in mixtures 10 and 11 which suggests a decrease in strength with increasing CCA content.

At 28 days of age most CCA concretes exhibited reduced strengths when compared with the two control mixtures. Mixtures 3, 7 and 12 were comparable with the controls having equal or slightly greater strengths. When comparing the "coarse" mixtures (10-16) to the "as received" mixtures (3-9), there is not a strong correlation between strengths. At 28 days of age most CCA concretes exhibited only slightly lower strengths than the control mixtures. When comparing source concrete strengths for the "coarse" group there is a more noticeable correlation between strength of new concrete versus source concrete strength when comparing mixture 11 to mixtures 12 and 13. Yet this correlation diminishes when comparing 14 made with the 5000 psi (34.5 MPa) source concrete to mixture 13. There is a correlation between percentage of replacement and strength when comparing mixtures 3 to 4 and mixtures 10 to 11.

At 90 days of age most CCA concretes exhibited reduced strengths when compared with the two control mixtures. Mixtures 3, 7 and 12 were comparable with the controls having equal or slightly greater strengths. When comparing the "coarse" mixtures (10-16) to the "as received" mixtures (3-9), the coarse mixtures tended to have slightly lower strengths on average. When comparing source concrete strengths for the "coarse" group there is a stronger correlation between strength of new concrete versus source concrete strength. Strengths increased for all mixtures as

source concrete strength increased. There is a correlation between percentage of replacement and strength when comparing mixture 3 to 4 and mixture 10 to 11. However the opposite occurred for mixtures 7 and 8 as strength increased with increasing CCA content.

The data from this research appears to be somewhat scattered. There are some correlations between source concrete strength and CCA strength but this was not always the case. There does also appear to be a correlation between percentage of replacement and strength as most results do show a decrease in strength as CCA content increases. Another notable observation is the substantial decrease in strengths for almost all the CCA concrete mixtures made from the “uncontrolled” pile 1 aggregate (mixtures 9, 15 and 16). However, mixtures 15 and 16 also had the highest w/c values and this would have definitely played a major role in the decreased strengths.

Mark Reiner (2007) investigated the effects of recycled materials on concrete strength. The mixtures incorporated 50% and 100% RCA as coarse aggregate and had varying amounts of fly ash as well. Two mixtures were batched with 50% RCA replacement and 30% fly ash replacement. One mixture utilized a class F fly ash and the other a class C. Four mixtures were batched using 100% RCA replacement. Two mixtures used class F fly ash at 30 and 40% replacement, and the other two mixtures used class C fly ash at 30 and 40% replacement. All of these mixtures were compared with control mixtures that had the same amount of cementitious materials and natural virgin aggregates. The results of the 50% RCA mixtures showed that the strengths were all slightly lower for all age groups up to 56-days when compared with the control mixture. However, the strength trend showed more differences at early age as opposed to later age strength.

The 100% RCA mixtures were based on a CDOT Class-B concrete mixture. These mixtures were tested up to 90-days of age as opposed to the 50% RCA mixtures which were only tested to 56-days. The compressive test results showed that strengths of all mixtures made with RCA were lower than the control mixture. However, three out of the four mixtures did meet the CDOT Class-B required strength of 3000 psi (21.4 MPa) at 28-days. Although tables were not provided indicating results for the 100% RCA mixtures (only graphs), an easy comparison of the 50% RCA and 100% RCA graphs shows that as RCA content increases, strengths decrease.

2.4.4.2 Permeability

According to information provided in ACI 555R (2001), the permeability of concretes incorporating RCA with w/c values in the range of 0.5 to 0.7 will be on the order of two to five times higher than normal concretes. This may be offset by lowering the w/c value by 0.05 to 0.10. Results reported by Mirjana Malešev et al (2010) support this claim. They found that as RCA content increased the water absorption increased when hardened specimens were tested at 28-days of age. The w/c for these mixtures was within the range of 0.5 to 0.7 as well. The ASTM procedure, if followed, was not reported in this study.

Results of rapid chloride ion penetrability studies performed by Karthik Obla et al (2007) showed a general trend of increasing permeability with increasing CCA content. They investigated the effects of crushed concrete aggregate (CCA) on hardened concrete properties. Multiple concrete mixtures with different replacement levels of CCA ranging from 10 to 100% were developed and tested. The types of CCA used also varied and were categorized based on original design strength of the source concrete. Four types of CCA were used; 1000, 3000, 5000 psi (6.9, 20.7 and

34.5 MPa) and an “uncontrolled” combination. The 1000, 3000 and 5000 psi (6.9, 20.7 and 34.5 MPa) CCA came from known sources and was in essence “controlled” with regards to aggregate properties. The uncontrolled CCA was labeled as “pile 1” because it came from the general recycled concrete pile at the batch plant and is a general mixture of various recycled concrete. Two different methods of incorporating the CCA into the mixture were used; one included the CCA in an “as received” state from the crusher and the other used pre-designated “Coarse” CCA aggregate. The “as received” CCA had a percentage of fine aggregate included within and therefore percentages of virgin fine aggregate were subtracted from the mixture design. If the “Coarse” method was used, only a percentage of the coarse aggregate was replaced. Table 2.13 summarizes the results from the RCP testing which was performed at 90-days of age.

Specimens were prepared and tested for rapid chloride ion penetrability according to ASTM C 1202. These results show that as CCA content increases, so does the permeability. The lowest results did come from the 10 and 20% total replacement mixtures 3 and 4 which were made from 1000 psi (6.9 MPa) source CCA. What is interesting is that as the source concrete strength increased, the permeability increased as can be seen in mixtures 5, 6 and 7 when compared with mixture 4. When comparing the 20 and 30% total replacements with 3000 psi (20.7 MPa) source concrete there is a significant increase in permeability with a 10% increase in CCA content. Although mixture 9 was made with the general Pile-1 CCA, this concrete exhibited lower permeability than both control mixtures at 20% replacement. These trends of increasing permeability with increasing source strength and CCA content was not repeated for mixtures designed with only coarse aggregate replacements. When comparing mixtures 10 and 11 the permeability decreased as CCA content increased. All mixtures designed with only coarse aggregate replacements showed significantly higher permeability's than those designed with

total replacements. This would suggest a general trend of increasing permeability with increasing RCA content. What can also be seen from these results is that it is difficult to produce a dense concrete when w/c values as high as these are used:

Table 2.13 RCPT Test Results of Concretes Made with Varying Amounts of CCA, Karthik Obla et al (2007).

Mixture ID		Source Concrete Strength (lb/in ²)	w/c	RCPT Test Results (Coulombs)	Permeability Class
1	Control-1	N/A	0.58	3618	Moderate
2	Control-2	N/A	0.58	3424	Moderate
3	10% T-CCA	1000	0.55	2970	Moderate
4	20% T-CCA	1000	0.59	2984	Moderate
5	20% T-CCA	3000	0.58	3936	Moderate
6	20% T-CCA	3000	0.58	3316	Moderate
7	20% T-CCA	3000	0.58	3683	Moderate
8	30% T-CCA	3000	0.59	4276	High
9	20% T-CCA	Pile 1	0.58	3232	Moderate
10	50% C-CCA	1000	0.55	5402	High
11	100% C-CCA	1000	0.52	5187	High
12	100% C-CCA	3000	0.59	6248	High
13	100% C-CCA	3000	0.59	5036	High
14	100% C-CCA	5000	0.57	4729	High
15	100% C-CCA	Pile 1	0.68	6201	High
16	100% C-CCA	Pile 1	0.69	6033	High

Gholamreza Fathifazl et al (2009) examined the effects of RCA on the hardened properties of concrete. As previously stated, for this particular study a new mixture design was developed to incorporate the existing mortar content within the

RCA. This new method accounts for this preexisting mortar in the design process and subtracts this amount from the new cementitious quantities. This new method of design is called the Equivalent Mortar Volume method (EMV). Several mixtures were developed from two RCA sources and compared with a similar control mixture composed of normal virgin aggregates and PC. To compare the significance of this new design method two separate batches were designed and mixed, one using the EMV method and the second using the traditional ACI method. Chloride penetration tests were performed on each mixture using the acid bulk diffusion test according to ASTM C 1556. They determined that the chloride diffusion coefficients for the RCA mixtures, regardless of design method were higher than that of the control specimens made from natural aggregates. When comparing the two different design methods, the EMV designed specimens had higher coefficients (on the order of 8 %) than the ACI designed specimens.

Mark Reiner (2007) investigated the effects of recycled materials on concrete permeability. Four mixtures incorporating 100% RCA as coarse aggregate and varying amounts of fly ash were tested. Two mixtures used class F fly ash at 30 and 40% replacement, and the other two mixtures used class C fly ash at 30 and 40% replacement. All of these mixtures were compared with control mixtures that had the same amount of cementitious materials and natural virgin aggregates. Testing was conducted at 14, 28 and 90 days of age. The results showed that at 14 days, all four mixtures with RCA had significantly higher results (charge passed). However, the results at 28 days of age showed different results. The two mixtures made with class-F fly ash had surpassed the control mixture and achieved lower ratings at 28-days. The two mixtures with class-C fly ash still achieved higher results. At 90 days of age, all but one RCA mixture achieved better results than the control mixture. The two mixtures with class-F fly ash actually achieved 31 to 36% improvements when compared with the control at 90-days.

2.4.4.3 Freeze-Thaw Durability

According to ACI 555R (2001), most studies performed on the freeze-thaw resistance of concretes made with RCA show little if any difference when compared to normal concretes made with natural aggregates. They did mention several Japanese studies which indicated a decrease in freeze-thaw resistance for concretes made with RCA but also noted that the studies performed in Japan used much lower quality of source concrete as compared to studies from other sources.

Karthik Obla et al (2007) performed freeze-thaw durability tests on concretes containing various replacement amounts of aggregate with crushed concrete aggregate (CCA). Three concrete mixtures with different replacement levels of CCA were developed and tested as well as a control mixture composed of natural aggregates. Two mixtures contained 20% CCA and one mixture contained 100% CCA for coarse aggregate. The types of CCA used also varied and were categorized based on original design strength of the source concrete. Two types of CCA were used; 1000 and 3000 psi (6.9 and 20.7 MPa). The 1000 and 3000 psi (6.9 and 20.7 MPa) CCA came from known sources and was in essence “controlled” with regards to aggregate properties. Two different methods of incorporating the CCA into the mixture were used; The first two mixtures included the CCA in an “as received” state from the crusher and the third used pre-designated “Coarse” CCA aggregate. The “as received” CCA had a percentage of fine aggregate included within and therefore percentages of virgin fine aggregate were subtracted from the mixture design. If the “Coarse” method was used, as was done for the third mixture, only a percentage of the coarse aggregate was replaced. Two beam specimens were developed for each mixture and tested at 56-days of age according to ASTM C 666 procedures. Table 2.14 summarizes the results from the freeze-thaw durability tests.

Mixture #4 had the highest air content for all mixtures but the air contents for the other three mixtures were well within reasonable limits to resist the freezing and thawing cycles. Both specimens from mixtures #2 and #3 that were mixed with the CCA in "as received" condition failed during testing. Both specimens from mixture #2 failed (i.e. the dynamic modulus fell below 60%) before completion of the test at 107 and 190 cycles respectively. It was also noted that visible cracks could be seen in these specimens. Both specimens from Mixture #3 performed better than Mixture #2 specimens but also failed before completion of the test at 243 and 300 cycles respectively. No visible deterioration was noted in these specimens however. Both specimens from Mixture #4 performed well. These mixtures had 100% CCA as coarse aggregate and the failure of the previous two mixtures was associated with the CCA fines content that was not present in Mixture #4.

Table 2.14 Average Results of Rapid Freeze-Thaw Testing on Concretes Made with CCA, Karthik Obla et al (2007).

Mixture ID		Source Concrete Strength (lb/in ²)	w/c	Air Content (%)	Durability Factor	Mass Loss (%)
1	Control	N/A	0.45	6.4	92	0.52
2	20% T-CCA	1000	0.45	4.8	13	0.18
3	20% T-CCA	3000	0.45	5.6	9	0.73
4	100% C-CCA	3000	0.45	8.5	89	1.23

Gholamreza Fathifazl et al (2009) performed freeze-thaw durability tests on concretes with 100% RCA as coarse aggregate. As previously stated, for this particular study a new mixture design was developed to incorporate the existing

mortar content within the RCA. This new method accounts for this preexisting mortar in the design process and subtracts this amount from the new cementitious quantities. This new method of design is called the Equivalent Mortar Volume method (EMV). Two mixtures were developed from one RCA source and compared with a similar control mixture composed of normal virgin aggregates and PC. To compare the significance of this new design method two separate batches were designed for the RCA mixtures, one using the EMV method and the second using the traditional ACI method. Freeze thaw durability testing was performed on each mixture using procedure A of ASTM C 666. It is assumed these specimens were tested at 28-days of age as this information was not provided. The durability factors (DF) for all mixtures, including the control were above 90%. The specimens designed by the new EMV method performed slightly better (3.3% higher DF) than the specimens designed by the traditional ACI method. They attributed this difference to the lower total mortar content of the EMV designed specimens.

Reiner (2007) investigated the effects of RCA content on the freeze-thaw durability of concretes. Four mixtures incorporated 100% RCA as coarse aggregate and had varying amounts of fly were tested. Two mixtures used class F fly ash at 30 and 40% replacement, and the other two mixtures used class C fly ash at 30 and 40% replacement. All of these mixtures were compared with control mixtures that had the same amount of cementitious materials and natural virgin aggregates. The mixtures were tested according to ASTM C 666 procedures. Although not reported, it is assumed these mixtures were tested at 28 days of age due to the inclusion of high amounts of fly ash. After 300 cycles of testing the RCA mixtures performed well. All four mixtures performed better than the control mixture and had durability factors above 85.

2.4.4.4 Alkali-Silica Reactivity (ASR)

The potential for ASR when using RCA as aggregate should raise the same, if not more concern to the designer when compared to natural aggregates. The potential for ASR in RCA depends greatly on the source aggregate and this information is not always known. If the source concrete information is available then a simple review of the aggregate used will probably determine whether or not ASR could be a problem in the new concrete. If the source concrete information is not available then caution should be exercised and testing should be performed on the RCA to determine potential for ASR. The source concrete aggregate may have tested positive for ASR but low alkali cement or other techniques could have been used to reduce this potential. The effects of any mitigation techniques used in the source concrete are no longer applicable once the concrete is crushed and used as RCA.

2.4.5 Summary

This literature review showed that there can be neutral or negative effects from using RCA as aggregate in a concrete. Neutral (or non-detrimental) effects are actually a good quality when it comes to aggregates. Aggregates are by and large a filler material in a concrete and do not contribute to strength until high strengths are required. Therefore, it would actually be uncommon or untypical to find a lot of supporting information that would suggest RCA increases a concrete's strength. Most information found in regards to RCA has shown neutral effects but in some cases the effects were detrimental or slightly beneficial.

It appeared that when researchers took care in choosing a reputable source concrete for use as RCA, and performed testing as if the RCA were any other natural aggregate, the results were positive and less scattered. The most critical aspect of

using RCA in a concrete is treating the RCA as an “unknown” aggregate source. RCA fines also appear to be attributed to negative results in concrete. All properties of concrete decreased as fines content increased from slump to compressive strength and durability. The compressive strength in general may decrease or remain the same but at best, will only increase slightly when compared with natural aggregate concrete. This is reasonable because once a concrete is crushed and recycled one cannot expect to achieve beneficial effects because the old concrete was a high strength concrete. There also tends to be more scattering of results if concretes are made from a general stock pile of recycled concrete as opposed to a known source concrete.

Although there wasn't a strong correlation between source concrete strength and new RCA concrete strength, there was minor evidence that suggested higher strength source concretes would produce higher strength RCA mixtures, when compared with other RCA mixtures made from lesser strength source concretes. This was probably due to the mortar still attached to the aggregates and one can expect that the higher strength concretes had a better bond between aggregate and paste and would therefore provide a stronger ITZ between the old mortar and new. However, the bond between the aggregate and old paste would still be the bond in question, not the new bond between old and new paste. At best, the bond between old and new paste can only be as strong as the bond between old paste and aggregate, since this will remain the weakest link. The freeze-thaw resistance of concretes made with RCA will most probably decrease. This decrease in freeze-thaw resistance does not appear to be correlated with amount of RCA content however but more on the fines content. The permeability of concretes made with RCA are expected to increase with increasing RCA content, regardless of fines. Other durability factors such as ASR may increase or decrease and this depends greatly on the source concrete and aggregate.

2.5 Expectations

This research involves testing concretes made with varying amounts of combined recycled materials. These replacements will be done together on all three ingredients equally. The combined effects that GGBFS, RCA and waste glass will have on concrete is unknown and can only be speculated based on the observed and reported effects that each alone have had on concrete. Since aggregate volume is much higher than paste volume it would be assumed that the aggregate qualities would have the most pronounced effect. However, the cementitious paste is the heart of the mixture and the GGBFS effects may govern. An attempt at theorizing the combined effects is warranted and these effects will be limited to those that are being tested for this research program.

The workability of a concrete was shown to increase with increasing GGBFS and decrease with increasing RCA and waste glass. Therefore, it can be assumed that since two of these materials decrease workability and only GGBFS increases workability that the total workability will decrease as recycled materials content increases. The AEA dosage of GGBFS mixtures tended to increase with increasing content. The AEA requirements of RCA also tended to increase but not as significantly and this was related to fines content. The air contents of waste glass mixtures did not appear to be effected. Therefore, it can be assumed that the combined effect will be a decrease in air contents for increasing recycled materials when AEA dosage is held constant.

The strength of concretes made with GGBFS tend to increase up to a certain percentage replacement and then decrease thereafter. The use of RCA tends to decrease a concrete mixtures strength with increasing content. The use of waste glass also tended to decrease a concretes strength with increasing content. Therefore, with two ingredients that decrease strength and one ingredient that increases to a certain

degree, it can be assumed that as recycled materials increase a trend of decreasing strength will be seen. Strength gains will probably be slower for increasing recycled contents due to the GGBFS.

The permeability of concretes made with GGBFS significantly decreased with increasing GGBFS content. The permeability of concretes made with RCA as coarse aggregate tended to increase with increasing content. The use of waste glass was shown to increase the permeability with increasing content. Whether or not the GGBFS can offset the increased permeability caused by the recycled aggregates will be interesting. However, with two ingredients that increase and only one that decreases permeability it will be assumed that permeability will increase for increasing recycled content. This seems reasonable since paste volume is significantly less than aggregate volume. One would think that the effects that the recycled aggregate have on the permeability will govern. However, the cementitious paste is the area where water penetrates and the GGBFS content may prove to govern this area. The freeze thaw durability of concretes made with GGBFS may decrease as replacement levels increase. The freeze thaw durability of concrete was also shown to decrease as RCA content increased as well as glass content when used as coarse and fine aggregates respectively. Therefore, it is assumed that as recycled materials content increases the freeze-thaw durability will decrease.

The alkali-silica reactivity of concretes made with glass aggregate has been shown to increase as content increases. The size of the glass particles will also play a major role. Since the glass aggregate used for this research is a mixed colored glass and is a 3/8 inch minus aggregate, the potential for ASR is real. The ASR potential of the RCA is unknown. The use of GGBFS has shown to decrease the ASR expansions in concretes made with reactive aggregates and may help this from occurring. It is expected that concrete mixtures with GGBFS will not be affected by ASR, but the concrete mixture with only recycled aggregates and PC will undergo ASR expansion.

3. Problem Statement

3.1 Statement

Today's society is changing. Gone are the days of excess and care-free living. Talk of the environment is on everyone's agenda, and global warming is something even children know about. Recycling has become a common practice not only in households across America, but in today's industry as well. Most products made today consist of at least some form of recycled material, whether it's the packaging it comes in or the product ingredients themselves. Energy consumption is another area of concern due to the increased awareness of global warming and the planets limited supply of natural fuel resources. It is for these reasons that the potential benefits of using recycled materials in concrete was chosen for the topic of this research.

Concrete is the most widely used construction material in the world. In residential construction the foundations on which houses, apartment buildings, town houses, etc. rest upon will most undoubtedly be made of concrete. This foundation may be entirely formed and placed or it may be a composite structure composed of cinder blocks and placed concrete. Sky scrapers and other larger commercial or residential buildings also have foundations of concrete, but unlike smaller residential structures these larger building may be made primarily of concrete. Concrete has been used to build many of the roadways and bridges throughout the world, not to mention many other civil structures such as dams and waterways. The importance of concrete in today's growing infrastructure is obvious. The use of concrete will only increase in the future, and the materials and energy needed to fulfill this demand are becoming more difficult to obtain.

The primary materials needed to produce concrete today consist of portland cement, aggregates and water. Cement is the most costly component of concrete. Unlike aggregates, cement cannot simply be mined from the earth. Cement

production is not only expensive but also produces large amounts of pollution. It is estimated that for every ton (metric ton) of cement produced an equal amount of carbon dioxide is emitted to the atmosphere (World Business Council for Sustainable Development, 2002). The aggregates used in concrete must be quarried from the earth and processed before being suitable for use. Aggregate quarries are not only destructive from a natural beauty standpoint, but also require substantial amounts of energy. An aggregate must also have suitable inert qualities before being used in concrete. Not every rock and gravel source is acceptable. These quality aggregate sources have diminished over the years and finding acceptable aggregates is becoming difficult. Concrete used for a particular job is not a material that can be shipped from great distances and concrete batch plants typically need to be located somewhere nearby. Economically, the aggregate sources must also be located close to the batch plant as well. Therefore, quality and proximity are key components to a concrete batch plant. The focus of this research is to determine the potential benefits of replacing portland cement and the aggregates of a concrete with recycled materials.

The purpose of this thesis is to design and batch concrete mixtures with varying percentages of recycled materials. To fully understand the effects that recycled materials have on the fresh and hardened concrete properties the percentages ranged from a 25% replacement to a complete 100% recycled material concrete. The compressive strength of a concrete is its most important property. Concrete used today must be able to carry the loads it is subjected to and must be durable enough to withstand harsh environments. The ultimate strength, permeability and freeze-thaw durability was examined for all mixtures as well as the fresh properties. Alkali-silica reactivity was also investigated for this research. The primary benefit of this research is to examine the positive and negative effects of using recycled materials in a concrete mixture, and determine at what point these materials no longer become beneficial.

4. Experimental Plan

4.1 Design Summary

The purpose of this research program was to investigate the potential benefits of using recycled materials in concrete mixtures. The three main ingredients that were subject to replacement with recycled materials included cement, fine and coarse aggregates. This research began by first obtaining the recycled materials and then performing tests to determine their physical properties. Recycled concrete was obtained to be used as a coarse aggregate replacement. Waste glass was obtained to be used as a fine aggregate replacement and ground granulated blast furnace slag (GGBFS) as a cement replacement.

In order to investigate the effects these recycled materials would have on the fresh and hardened properties of the concrete, six mixtures were designed and batched. These mixtures were designed based on a Class D, Colorado Department of Transportation (CDOT) concrete mixture. A Class D CDOT concrete is designated as a medium strength structural concrete that can be used in multiple applications including bridge decks. Certain requirements for a Class-D CDOT concrete are shown in Table 4.1. It should be noted that these mixtures were only “loosely” based on a Class-D mixture because the use of GGBFS and replacement amounts used for this research are prohibited. Recycled concrete and waste glass are also prohibited. However, the cementitious content, percentages of coarse and fine aggregates, w/c and air content guidelines were all followed for these mixtures.

The total cementitious contents and w/c were held constant for all mixtures. All mixtures had similar ratios of coarse and fine aggregates as well. The research began by first batching a mixture with 100% replacement of aggregates and cement with recycled aggregates and GGBFS. A second mixture was batched at the same time which had 100% of the aggregates replaced with recycled aggregates, but had

100% PC. These two mixtures were batched first to investigate any potential adverse effects between the materials and any effects on fresh properties that may develop from batching. The next stage of research involved designing and batching intermediate replacement mixtures ranging from 25 to 75% replacements. A control mixture composed of normal aggregates and 100% PC was also batched to be used for comparison.

Table 4.1 Colorado Department of Transportation (CDOT) Class-D concrete specifications and field requirements

28-Day Compressive Strength	Total Cementitious Range	Air Content Range	Maximum Water to Cementitious	Minimum Amount of Coarse Agg.
<i>(lb/in²)</i>	<i>(lb/yd³)</i>	<i>(percent)</i>	<i>(ratio)</i>	<i>(percent)</i>
4500	615 – 660	5 – 8	0.44	55

4.2 Material Properties

4.2.1 Ground Granulated Blast Furnace Slag (GGBFS)

The GGBFS was obtained from Lehigh, Heidelberg Cement Group located in Concord, California. Because this research involved developing a concrete mixture that had a 100% replacement of cement with GGBFS, a Grade 120 slag was chosen because of its higher reactivity compared to Grade 100. The GGBFS was tested in accordance with ASTM C 989 and the results of this testing are shown in Table 4.2.

Table 4.2 Lehigh Grade 120 GGBFS Physical and Chemical Properties

ALLCEM GGBFS Grade 120			
Chemical and Physical Properties		Test Results	ASTM C 989 Specifications
Sulfur Trioxide (SO ₃)	(%)	2.192	4.0 Max
Sulfur Sulfide (S)	(%)	0.560	2.5 Max
Blaine Fineness	cm ² /g	5240	-----
Percent Retained on 325 Mesh	(%)	0.2	20 Max
Air Content of Slag Mortar	(%)	6.3	12 Max
Specific Gravity		2.93	-----
Slag Activity Index			
7-Day Individual	(%)	134	90 Min
28-Day Individual	(%)	138	110 Min
Compressive Strength			
7-Day Reference Cement	(lb/in ²)	4890	-----
28-Day Reference Cement	(lb/in ²)	6260	5000 Min
7-Day Slag + Ref. Cement	(lb/in ²)	6560	-----
28-Day Slag + Ref. Cement	(lb/in ²)	8660	-----

4.2.2 Portland Cement

The cement used for this research was a Type-I-II portland cement. The cement was supplied by Holcim Cement Company, located in Florence, Colorado. The cement was tested in accordance to ASTM C 150 and the results are shown in Table 4.3.

Table 4.3 Holcim Type-I-II Cement, Physical and Chemical Properties

Holcim Type-I-II Portland Cement			
Chemical and Physical Properties		Test Results	ASTM C 150 Specifications
SiO ₂	(%)	19.6	----
Al ₂ O ₃	(%)	4.7	6.0 max
Fe ₂ O ₃	(%)	3.2	6.0 max
CaO	(%)	63.4	----
MgO	(%)	1.5	6.0 max
SO ₃	(%)	3.4	3.0 max
CO ₂	(%)	1.4	----
Limestone	(%)	3.7	5.0 max
CaCO ₃ in Limestone	(%)	84.0	70 min
C ₃ S	(%)	59.0	----
C ₂ S	(%)	11.0	----
C ₃ A	(%)	7.0	8 max
C ₄ AF	(%)	10.0	----
C ₃ S + 4.75 C ₃ A	(%)	92.0	100 max
Loss of Ignition	(%)	2.6	3.0 max
Blaine Fineness	cm ² /g	414	2600 - 4300
Air Content of PC Mortar	(%)	6.3	12 max
Specific Gravity		3.15	----

4.2.3 (Virgin) Coarse and Fine Aggregates

Both the coarse and fine aggregates were obtained by University of Colorado Denver. Both these aggregates came from Bestway Concrete and sources located in the Colorado area. The material properties and gradations of these aggregates were performed by WestTest Laboratories located in Denver, Colorado. The coarse aggregate was tested in accordance to ASTM C 33 No. 57 and 67 Coarse Aggregate and meets these requirements. The sand was tested in accordance to ASTM C33 Fine Aggregate, and meets these requirements. These aggregates will be referred to as “UCD sand” and “UCD rock” for the remainder of this thesis. The material properties data and complete gradations for both aggregates are included in Appendix B.

4.2.4 (Recycled) Coarse and Fine Aggregates

The recycled concrete and waste glass used for replacement of aggregates was fully tested prior to use in a concrete mixture. All aggregate samples used for testing conformed to ASTM C 702 requirements. Table 4.4 summarizes the tests performed.

Table 4.4 Testing of Recycled Materials

Aggregate	Test Type Performed	ASTM Method
Recycled Concrete (Coarse Aggregate)	Specific Gravity & Absorption Capacity	ASTM C 127
	Sieve Analysis (Gradation)	ASTM C 136
	Unit Weight & Moisture Contents	ASTM C 29
Waste Glass (Fine Aggregate)	Specific Gravity & Absorption Capacity	ASTM C 128
	Sieve Analysis (Gradation)	ASTM C 136

4.2.4.1 Waste Glass as Fine Aggregate

The waste glass was obtained from Rocky Mountain Bottling Co., owned by Coors. The waste glass was used as a replacement for the UCD sand. The waste glass was produced mainly from beer bottles and consisted of various colors (clear, amber and green). The process of recycling bottles involves multiple crushers which decrease the size of the glass particles more after each crusher. The glass moves along a conveyer belt from one crusher to the next while vacuums and magnets remove any labels or other foreign objects. After all crushing has taken place the glass drops into a hopper and is ready for the furnace where it will be melted down. It is from this hopper that the glass for this research was taken. The glass was used "as received" from the recycling plant and no washing took place. Only larger metallic objects (i.e. batteries) were hand removed from the waste glass prior to use in the concrete mixtures. Testing was performed on the waste glass to determine the physical and chemical properties. Table 4.5 shows the properties of the waste glass as well as the UCD sand fine aggregate for comparison.

Table 4.5 Fine Aggregate Properties of Waste Glass and UCD Sand

Aggregate Property	Waste Glass	UCD Sand
Absorption Capacity, (%)	0.18	0.70
Specific Gravity	2.47	2.63
Fineness Modulus	4.08	2.74

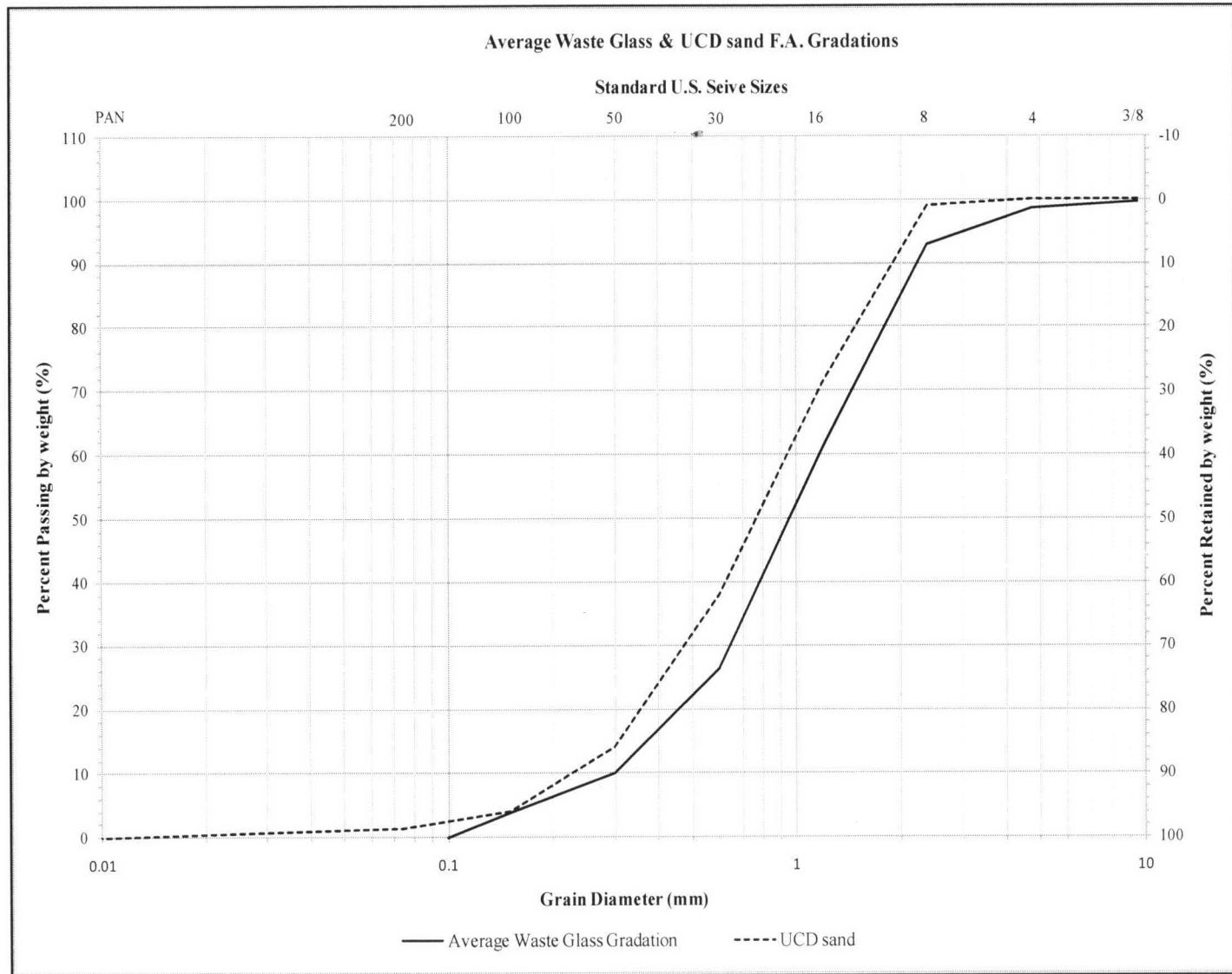
When comparing these two fine aggregates it can be seen there are similarities and differences in physical properties. The specific gravity (SG) of both aggregates

are similar. Although both aggregates have low absorption capacities, the waste glass is much lower. This can be expected because of the dense micro structure of the glass which doesn't have many open pores for water to be absorbed.

Three separate gradations were performed on the waste glass. Before performing these gradations, the waste glass had to be sampled properly. ASTM C 702, Standard Practice for Reducing Samples of Aggregate to Testing Size was followed for sampling. This process involved dumping all buckets of waste glass together and thoroughly mixing, the pile is repeatedly broken down into four separate piles. This process goes on until two suitable piles can be combined for a sieve analysis (i.e. the two combined piles must have a weight within the range for testing). The gradation curves for the waste glass and UCD sand are shown in Figure 4.1 for comparison. This waste glass plot is an average of three gradations performed. The gradations for all three specimens did not meet the requirements of ASTM C 136 due to excess materials retained on the 16, 30, and 50 sieves. These results were consistent with all three samples tested. Tables summarizing the results for all three samples tested are included in Appendix B.

The fineness modulus (FM) for the waste glass is much higher than the UCD sand. For workability reasons, the FM for a fine aggregate should typically be between 2.3 and 3.1 when used in concrete. However, the FM should not be used to compare two aggregates from different sources as this parameter is a crude measure of grading and two aggregates with similar FM can have very different gradation curves, (Mindess, 2003). Typically the higher the FM value the coarser the fine aggregate particle size. Therefore, a lower FM is better as this indicates finer sand and more particles will be available to help improve workability in a concrete mixture. Based on the test results, the waste glass is a coarser aggregate than the UCD sand and may cause workability issues.

Figure 4.1 Average Waste Glass & UCD Sand Gradations



4.2.4.2 Recycled Concrete as Coarse Aggregate

The recycled concrete was obtained from Oxford Recycling located in Denver, Colorado. The aggregate was taken from the large pile designated as $\frac{3}{4}$ inch (19 mm) nominal. The recycled concrete was picked up twice due to re-batching requirements, once in the winter and once in the following fall. Both times the aggregate was randomly shoveled into buckets from several locations around this large pile. The actual history behind the aggregate taken for this research is unknown. The recycled concrete is made from demolished roads, buildings, and other various concrete structures that have been removed from service. Most of the recycled concrete comes from roads. Testing was performed on the recycled concrete to determine the physical and chemical properties. Table 4.6 shows the properties of the recycled concrete as well as the UCD rock coarse aggregate for comparison. The specific gravity and absorption capacity values are the average of two tests while the dry rodded unit weight value is the result of one test only. The nominal maximum aggregate size (NMAS) is the result from three consistent tests.

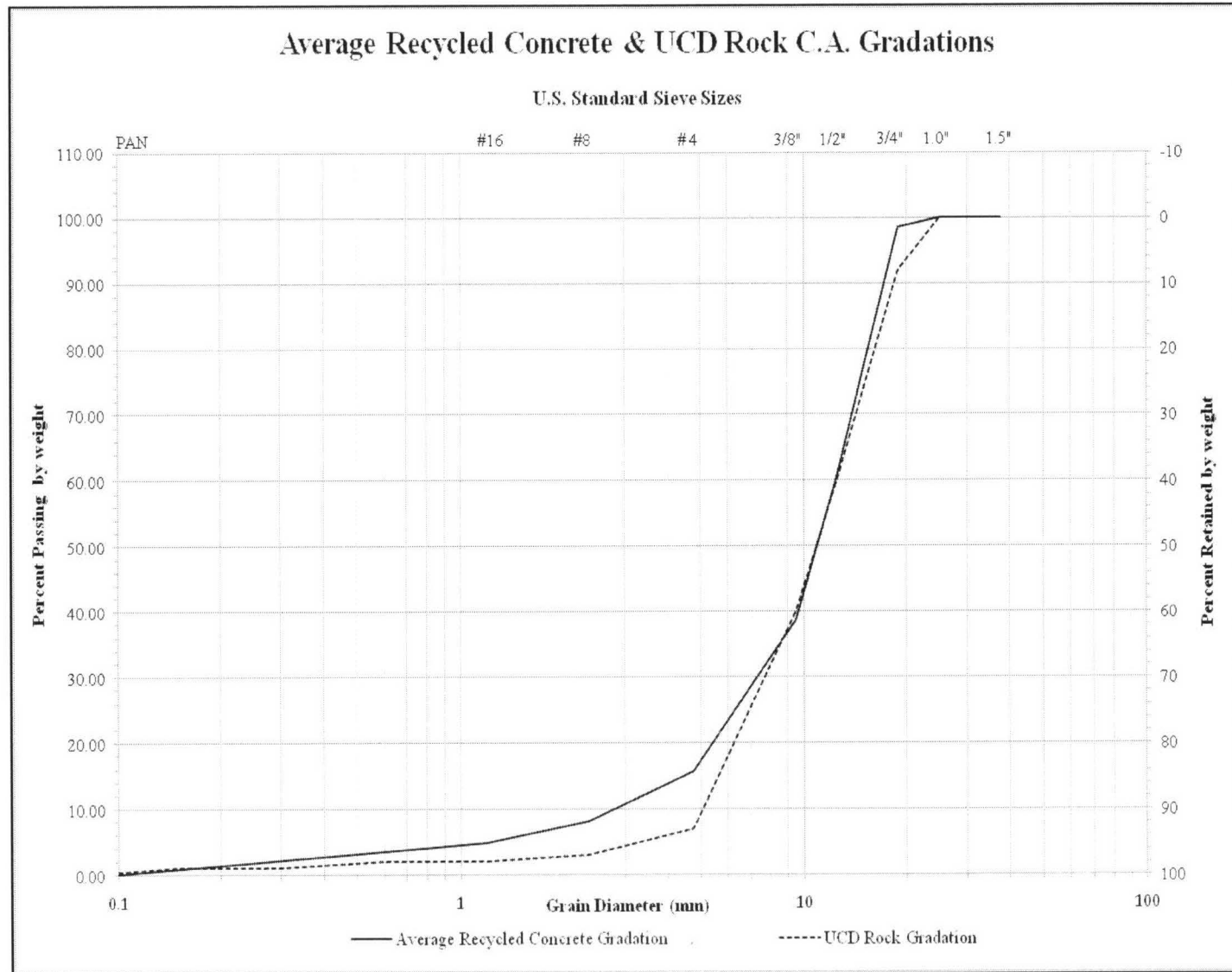
Table 4.6 Coarse Aggregate Properties of RCA and UCD Rock

Aggregate Property	Recycled Concrete	UCD Rock
Absorption Capacity, (%)	4.58	0.80
Specific Gravity	2.45	2.61
Dry Rodded Unit Weight, (lb/ft^3)	85.02	103
NMAS, (<i>inch</i>)	$\frac{3}{4}$	$\frac{3}{4}$

When comparing these two coarse aggregates one can see there are significant differences between them. The recycled concrete has a lower SG which correlates with the lower unit weight as well. The recycled concrete contained various sizes of aggregate with mortar still attached as well as numerous small chunks of pure mortar. Although cement has a higher SG than aggregates used in concrete, the lower unit weight and SG may be a result of entrapped and entrained air within the mortar coatings and mortar chunks. The recycled concrete comes from Colorado sources where entrained air would typically be required. The absorption capacity of the recycled concrete is considerably higher than the UCD rock aggregate. This is also caused by the mortar chunks and mortar coated aggregate. Cement is porous and will absorb water. These results are not uncommon as most reports on recycled aggregate indicate these same properties. This increase in absorption capacity will put a heavy demand on a concrete mixture with regards to water. When combined with a low w/c and coarse fine aggregate the workability may cause issues.

Three separate gradations were performed on the recycled concrete. Before performing these gradations, the recycled concrete had to be sampled properly. ASTM C 702, Standard Practice for Reducing Samples of Aggregate to Testing Size was followed for sampling. This process involved dumping all buckets of recycled concrete together and thoroughly mixing, the pile is repeatedly broken down into four separate piles. This process goes on until two suitable piles can be combined for a sieve analysis (i.e. the two combined piles must have a weight within the range for testing). The gradation curves for the recycled concrete and UCD rock are shown in Figure 4.2 for comparison. The recycled concrete plot is an average of three separate gradations. The gradations for all three specimens did not meet the requirements of ASTM C 136 due to excess materials retained on the number 4 (all three samples) and 8 (one sample only) sieves. Tables summarizing the results for all three samples tested are included in Appendix B.

Figure 4.2 Average RCA and UCD Rock Gradations



4.2.5 Chemical Admixtures

Chemical admixtures were used in all mixtures batched for this research to improve workability and to entrain air. Both the admixtures are also required to satisfy CDOT Class D mixture requirements. One air entraining admixture was used for all mixtures. Two different high range water reducing admixtures (HRWRA) were used.

4.2.5.1 Air Entraining Admixture (AEA)

Duravair AT-60 was used for all mixtures to achieve a target air content of 6.5% in the concrete. This air entraining agent is manufactured by W.R. Grace & Co, Connecticut. The chemical properties and material data sheets for Duravair AT-60 are included in Appendix B.

4.2.5.2 High Range Water Reducing Admixture (HRWRA)

A high range water reducing admixture was used for all mixtures batched during this research due to the low w/c and high water and workability demands of the recycled aggregates. Two separate HRWRA were used due to the performance of the first HRWRA which prompted a switch in products to achieve desired slumps. The first HRWRA used was DARACEM 19, manufactured by W.R. Grace & Co, Connecticut. The second HRWRA used was Eucon 37, manufactured by Euclid Chemical Company. The chemical properties and material data sheets for both the DURACEM 19 and Eucon 37 are included in Appendix B.

4.3 Mixture Designs

A total of six mixtures were designed for this study. Ultimately a total of nine mixtures were batched and tested during this research. The purpose of this research was to investigate the effects that recycled materials have on concrete and to ultimately design and test a concrete composed of entirely recycled materials. A mixture design matrix showing all six research mixtures is shown in Table 4.7.

Table 4.7 Concrete Mixture Design Matrix

Material		Mixture # & Mixture Identification					
		1	2	3	4	5	6
		CC (Control)	100-RA-C	100-RA-BF	25-RA-BF	50-RA-BF	75-RA-BF
Cement (Type I-II)	(lb/yd ³)	615	615	0	461	308	154
GGBFS (Grade 120)	(lb/yd ³)	0	0	615	154	308	461
Cementitious (Cement + GGBFS)	(lb/yd ³)	615	615	615	615	615	615
Coarse Agg. (UCD rock)	(lb/yd ³)	1646	0	0	1200	800	400
Fine Agg. (UCD sand)	(lb/yd ³)	1324	0	0	1001	649	316
Coarse Agg. (Recycled Concrete)	(lb/yd ³)	0	1646	1646	400	800	1200
Fine Agg. (Waste Glass)	(lb/yd ³)	0	1139	1103	313	608	887
Water	(lb/yd ³)	246	246	246	246	246	246
W/CM	(ratio)	0.40	0.40	0.40	0.40	0.40	0.40
Air Content	(%)	6.5	6.5	6.5	6.5	6.5	6.5
AEA	(fl oz/cwt)	1.5	1.5	1.5	1.5	1.5	1.5
HRWRA (*Dosage Varied)	(fl oz/cwt)	*	*	*	*	*	*

The total cement content for all mixtures was chosen based on a CDOT Class-D concrete mixture as well as the w/c. The cementitious content of 615 lb/ft³ (365 kg/m³) was chosen and is the lowest amount acceptable. The w/c value of 0.40 chosen was the lowest acceptable level and with the use of HRWRA this value seemed reasonable to work with. All mixtures incorporated AEA to achieve the designed 6.5% air content. This value was also chosen based on the CDOT Class-D requirements. One brand of AEA was used for all mixtures and the dosage was held constant at 1.5 fl oz/cwt (1.0 ml/kg). A HRWRA was used on all mixtures to achieve workability. The dosage for the HRWRA used varied depending on mixture and in several cases the dosage used was well in accordance of the manufacturer's recommendations.

The absolute volume method was used to design these mixtures. By default a coarse aggregate weight is chosen as well as a cementitious content and w/c. The fine aggregate content is determined by default as the remainder of a cubic yard. Therefore, when determining aggregate quantities for mixtures with both normal and recycled aggregates the coarse aggregate was split up by weight and the fine aggregate was split up by volume. Coincidentally the SG of both coarse aggregates (recycled concrete and UCD rock) and both fine aggregates (UCD sand and waste glass) are similar and therefore the percentages by weight are close to the percentages by volume.

Mixture #1 designated as CC (Control) was the control mixture for this research. This mixture contained 100% UCD aggregates and 100% Type-I-II portland cement as the cementitious content. This mixture had 615 lb/ft³ (365 kg/m³) of cementitious which was composed entirely of PC. The total aggregate weight was distributed to 55% coarse aggregate and 45% fine aggregate. By volume the coarse and fine aggregates occupied 37 and 30% respectively for a combined total volume of

67%. The paste volume (PC, water and air) occupied the remaining 33% of the mixture.

Mixture #2 designated as 100-RA-C was designed to investigate the effects of recycled aggregates on a concrete made with 100% PC as cementitious. This mixture had 615 lb/ft³ (365 kg/m³) of cementitious which was composed entirely of PC. This mixture contained 100% recycled concrete and 100% waste glass as the coarse and fine aggregates respectively. The cementitious content was a Type-I-II PC. The total aggregate weight was distributed to 59% coarse aggregate and 41% fine aggregate. By volume the coarse and fine aggregates occupied 40 and 27% respectively for a combined total volume of 67%. The paste volume (PC, water and air) occupied the remaining 33% of the mixture.

Mixture #3 designated as 100-RA-BF was designed based on a 100% replacement of content with recycled materials. This mixture had 615 lb/ft³ (365 kg/m³) of cementitious which was composed entirely of GGBFS. This mixture contained 100% recycled concrete and 100% waste glass as the coarse and fine aggregates respectively. The total aggregate weight was distributed to 60% coarse aggregate and 40% fine aggregate. By volume the coarse and fine aggregates occupied 40 and 27% respectively for a combined total volume of 67%. The paste volume (GGBFS, water and air) occupied the remaining 33% of the mixture.

Mixture #4 designated as 25-RA-BF was designed based on a 25% replacement of content with recycled material. This mixture had 615 lb/ft³ (365 kg/m³) of cementitious materials. The cementitious content was split up by weight into 25% GGBFS, which totaled 154 lb/ft³ (91.4 kg/m³) and 75% PC which totaled 461 lb/ft³ (274 kg/m³). The total aggregate weight was distributed to 55% coarse aggregate and 45% fine aggregate. The coarse aggregate was then split into 25 and 75% recycled concrete and UCD rock by weight respectively. By default, the fine

aggregate was split into 23.8 and 76.2% waste glass and UCD sand by weight respectively. By volume, the recycled concrete and waste glass combined to occupy 25.7% of the total aggregate volume. By volume the combined coarse and fine aggregates occupied 37 and 30% respectively for a total of 67%. The paste volume (PC, GGBFS, water and air) occupied the remaining 33% of the mixture.

Mixture #5 designated as 50-RA-BF was designed based on a 50% replacement of content with recycled material. This mixture had 615 lb/ft^3 (365 kg/m^3) of cementitious. The cementitious content was split up by weight into 50% GGBFS and PC, each totaling 307.5 lb/ft^3 (182.4 kg/m^3). The total aggregate weight was distributed to 55% coarse aggregate and 45% fine aggregate. The coarse aggregate was then split into 50% recycled concrete and UCD rock by weight respectively. By default, the fine aggregate was split into 48% waste glass and 52% UCD sand by weight respectively. By volume, the recycled concrete and waste glass combined occupied 50.9% of the total aggregate volume. By volume the combined coarse and fine aggregates occupied 37.5 and 30% respectively for a total of 67.5%. The paste volume (PC, GGBFS, water and air) occupied the remaining 32.5% of the mixture.

Mixture #6 designated as 75-RA-BF was designed based on a 75% replacement of content with recycled material. This mixture had 615 lb/ft^3 (365 kg/m^3) of cementitious. The cementitious content was split up by weight into 75% GGBFS, which totaled 461 lb/ft^3 (274 kg/m^3) and 25% PC which totaled 154 lb/ft^3 (91.4 kg/m^3). The total aggregate weight was distributed to 57% coarse aggregate and 43% fine aggregate. The coarse aggregate was then split into 75 and 25% recycled concrete and UCD rock by weight respectively. By default, the fine aggregate was split into 73.7 and 26.3% waste glass and UCD sand by weight respectively. By volume, the recycled concrete and waste glass combined occupied 75.7% of the total aggregate volume. By volume the combined coarse and fine aggregates occupied 38

and 28.5% respectively for a total of 66.5%. The paste volume (PC, GGBFS, water and air) occupied the remaining 33.5% of the mixture.

4.3.1 Mixture Batching

All mixtures were batched following the guidelines stipulated by ASTM C 192 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory. The batching of the specimens for ASR testing is discussed in Chapter 5. Prior to batching the aggregate moisture contents were taken. To accomplish this all aggregates were dumped into a wheel barrow and mixed before obtaining a sample. This sample was taken the day before batching. After initial weight was taken the sample was placed in the oven for drying over night and weighed the following day just prior to batching. The aggregates in the wheel barrows were covered with a tarp overnight to help maintain moisture content.

The batching procedure was similar for all mixtures except for the addition of admixtures, which varied considerably. The AEA was always added to the water prior to batching. Two batching procedures were followed in regards to HRWRA dosage procedure during this research. The first procedure involved adding the HRWRA after all materials were in the mixer. The second procedure involved dividing the water into two equal buckets. The first half used contained the AEA and was allowed to mix with the aggregates for several minutes alone. The second half of the water contained the HRWRA and was dispersed after the cementitious contents were dumped into the mixture.

For all batching, the coarse aggregate and fine aggregate were placed into the mixer with half of the water (regardless of procedure this water contained the AEA) and allowed to mix for several minutes. Next, approximately 75% of the cementitious

was added and allowed to mix for a short time. Finally, the remaining water was added and if procedure #1 was followed, the HRWRA, and the mixture was allowed to mix for several minutes. If the second procedure was followed the remaining half of water was added with the HRWRA incorporated.

4.3.2 Curing

After preparing specimens for testing, all mixtures were placed in the humidity controlled room overnight. The humidity and temperature level of this room is held constant at 40% and 70 °F respectively. On the following day, all specimens were removed from the molds and submerged in water until time of testing.

4.4 Concrete Testing

Fresh concrete properties were examined immediately after batching. The fresh properties examined included slump, unit weight and air content. The hardened concrete properties were examined beginning one day after batching and continued through 90 days of age. The hardened concrete properties tested included compressive strength, rapid chloride ion penetrability, freeze-thaw durability and alkali-silica reactivity (ASR). Table 4.8 summarizes the testing performed, method and time frames.

Table 4.8 Fresh and Hardened Concrete Properties Tested

Fresh Concrete Tests	Standard Followed	Time of Testing
Slump	ASTM C 143	When Batched
Unit Weight	ASTM C 138	When Batched
Air Content	ASTM C 231	When Batched
Hardened Concrete Tests	Standard Followed	Time of Testing
Compressive Strength	ASTM C 39	1, 7, 28, 56, 90 Days (15-cylinders)
RCPT	ASTM C 1202	28, 56, 90 Days (6-cylinders)
Freeze-Thaw Durability	ASTM C 666	Begin at 28 Days (2-beams)
Alkali-Silica Reactivity (ASR)	ASTM C 1567	Begin at 2 Days (3-prisms)

5. Experimental Results

5.1 General

This chapter contains the presentation of the results and any significant observations noted during this research program. The beginning of this chapter begins briefly with an explanation of any adverse conditions encountered during the research program. The fresh concrete results are presented next followed by the hardened concrete results. The mixture designations from the mixture design matrix in Chapter 4 will be used throughout.

5.2 Problems with this Study

5.2.1 Re-Batch of Mixture #2 (100-RA-C) and Mixture #3 (100-RA-BF) Due to Consolidation Concerns.

When this study began the first two mixtures batched were the 100% recycled aggregate-cement mixture (mixture #2) and the 100% recycled aggregate-GGBFS mixture (mixture #3). The anticipated fresh concrete properties of these two mixtures was unknown at the time and it was assumed that HRWRA would be required since the w/c was 0.40 for all mixtures. No specific target slump was specified except that which would allow proper consolidation of the test specimens. The HRWRA used was DARACEM 19. The recommended dosage for the cementitious content of 615 lb/yd³ (365 kg/m³) was 60 ml (4.6 fl oz/cwt). After several minutes of mixing with all the ingredients including the HRWRA, it was clear that the slump would be negligible. For both mixtures the cementitious content clumped up into balls and would not separate and disperse. More HRWRA would have been used on the first mixture but there was only enough left to save for the second mixture being batched that day.

Both mixtures had negligible slumps (0 inches (mm)) and were very difficult to consolidate. There appeared to be un-hydrated cement and GGBFS clumps within the mixtures. The following day the molds were stripped and it was clear that these specimens may not give satisfactory results due to excessive honey combing around the outside of the cylinders as well as the freeze-thaw beams. It was decided to continue with the program and test these specimens assuming the honey combing was only present on the outside surface. Compressive strengths were quite low for both these mixtures but the expectations were not known and therefore it was unclear whether this was due primarily to the honey-combing or simply due to the recycled materials. However, at the time of the first RCPT testing it was impossible to create a tight seal around the specimens for both mixtures and permeability tests could not be performed. The freeze-thaw beams for both mixtures also deteriorated rapidly, within 30-40 cycles and it was decided to re-batch both mixtures.

Mixtures #2 and #3 were re-batched using a different HRWRA with much larger dosages. Eucon 37 was used and a different technique was applied regarding application of the HRWRA. Previously, on the first batching the HRWRA was added after all ingredients were in the mixer. For this re-batch it was decided to split the water into two equal amounts and add the air entrainment to the first bucket and allow this to mix with the aggregates and cement separately for several minutes. The second bucket contained three times the recommended dosage of HRWRA (180 ml.) and was added last. Although this method did break up the clumps of cementitious it was still evident that the slump would be negligible for both mixtures. More HRWRA was added (up 740 ml.) without results. Both mixtures only produced ½ inch slump at best but the new technique as well as new HRWRA proved to be useful as the consolidation was successful. Although difficult to consolidate, when the specimens were tapped on the sides the cementitious rose to the surface. All specimens looked great after removing the molds and no honey-combing existed.

5.2.2 Re-Batch of Mixture #2 (100-RA-C) Due to a Lack of Specimen Quantities

When Mixtures #2 and #6 were re-batched due to consolidation concerns mentioned previously, a mistake was made when counting test specimens. A total of 21 – 4 x 8 inch (102 mm x 203 mm) cylinders were required to perform the compressive testing as well as the RCPT testing. For Mixture #2 only 15 cylinders were cast (6 short) and for Mixture #6 only 18 cylinders were cast (3 short). It was decided that Mixture #6 would not need to be re-batched because this was the 100% GGBFS mixture and the strength gains were anticipated to be much slower and later age strength specimens were the most important. Therefore, only two cylinders were tested for 1, 7 and 28 days of age for Mixture #6. This is still acceptable according to ASTM procedures as only two cylinders are required. Mixture #2 was re-batched because this was a 100% cement mixture and 6 cylinders was thought to be a stretch when comparing data. However, these test specimens were still used and averaged with the Mixture #2 re-batch specimens. Two specimens were used for each compressive strength test and one specimen was used for each RCPT test. This combined for a total of 5 compressive strength cylinders for each age and 3 cylinder for each RCPT test age when combined for Mixture #2.

5.2.3 Freeze-Thaw Chamber

During the course of the durability testing the freeze-thaw chamber had several issues arise. The first set of freeze-thaw specimens to be tested included mixture #1, 2, 3 and 5. These mixtures were batched in early March of 2010. In May of 2010 the control beam fractured and the machine was shut down until a new control beam could be made. At the time of failure, mixture #2 and #3 specimens had already shown significant deterioration and as previously mentioned these mixtures were abandoned

and the specimens were discarded. Mixture #1 and #5 specimens had undergone approximately 56 cycles at the time of failure. According to ASTM C 666 procedures, if testing must be stopped for any reason the specimens should be stored in a frozen condition until testing can once again commence. This procedure was not followed and the test beams were stored in a dry condition until the new control beam was made and testing could continue. Approximately 5 weeks had passed before a new control beam was installed. Although a new control beam was installed, testing did not continue until late July of 2010 due to concerns regarding the calibration of the freeze-thaw chamber. It was believed that the machine was cycling too rapidly and the specimens were not completely thawing after each freeze cycle. Therefore, a total of 11 weeks had passed before testing continued, and during this time period the specimens were stored in the dry condition.

The second set of specimens to undergo freeze-thaw testing were mixtures #2, 2 (Re-Batch), 3, 4, and 6. These specimens were batched in November of 2010. Testing began in December of 2010 for these specimens. In January of 2011 a new control module was purchased for the machine. Although concerns were raised regarding the interruption of the freeze-thaw testing, it was decided that this new control module would be installed and testing was stopped. It was realized that the previous specimens tested were supposed to be stored in a frozen state. However, due to mistake made with previous specimens (mixture #1 and #5) it was decided to store these specimens the same way (in the dry) as the previous such that conditions would be identical. The new control module was installed and calibrated near the end of February and testing continued. The specimens had been stored in the dry for approximately 6 weeks. Although these problems appeared troublesome at first, in hind sight the specimens for all mixtures were now more closely related since both sets of testing had been interrupted and the specimens were stored in the dry.

5.3 Fresh Concrete Properties

The fresh concrete properties tested included slump, unit weight and air content. The results of these fresh concrete properties are included in Table 5.1.

Table 5.1 Fresh Concrete Properties

Mixture Identification		Slump (<i>inch</i>)	Air Content (<i>%</i>)	Measured Unit Weight (<i>lb/ft³</i>)
1	CC (control)	1.75	10.0	134.8
2	100-RA-C	0.50	5.2	139.2
3	100-RA-BF	0.50	4.6	140.8
4	25-RA-BF	8.25	6.4	138.6
5	50-RA-BF	0.50	7.0	140.8
6	75-RA-BF	3.00	7.5	134.1

5.3.1 Slump

As previously mentioned, there was no specific target slump required for these mixtures. However, in general a slump of 3 inches (76.2 mm) was believed to be a reasonable value to achieve consolidation on the test specimens. Due to the low w/c values on these mixtures, and the unknown effects of the combined recycled materials, a HRWRA was used for all mixtures and the dosage varied depending on mixture. Two different techniques were used when applying the HRWRA. Procedure #1 involved adding the HRWRA into the mix after all ingredients were in the mixer and after several minutes of mixing. Procedure #2 involved dividing up half of the

water and adding the HRWRA with half of the water at the end of the mixing and allowing the mixing to continue for several more minutes. An AEA was also used on all mixtures and this dosage remained constant at 20 ml (1.5 fl oz/cwt).

Mixtures #2 and #3 had the lowest slumps of only 0.5 inches (12.7 mm). Both of these mixtures had 100% recycled aggregates and differed only in the type of cementitious material. Mixture #2 was 100% PC and Mixture #3 was 100% GGBFS. These values are the values obtained from the re-batch of these mixtures. The original mixtures had negligible slumps (0 inch (0 mm)). The first batching of these mixtures contained 60 ml. (4.6 fl oz/cwt) of DARACEM 19 and the second batches contained up to 740 ml. (56.4 fl oz/cwt) of Eucon 37. The first batching used procedure #1 for addition of HRWRA. The second batching used procedure #2, yet after unsuccessful results additional HRWRA was dumped liberally into the mixer. These mixtures were difficult to work with yet the consolidation was successful on all re-batched test specimens.

Mixture # 5 had a slump of only 0.5 inch (12.7 mm). This mixture had 50% recycled aggregates and 50% natural aggregates. The HRWRA used was DARACEM 19 and the dosage was 60 ml (4.6 fl oz/cwt). Procedure #1 was followed for adding the HRWRA. Although this mixture only had a slump of 0.5 inch (12.7 mm) the workability was surprisingly decent. All test specimens were consolidated with only a little extra effort.

Mixture #1 which was the control mixture achieved 1.75 inch (44.5 mm) of slump. This mixture had 100% PC as well as 100% natural aggregates. DARACEM 19 was the HRWRA and a dosage of 60 ml (4.6 fl oz/cwt). was used. Procedure #1 was followed for adding the HRWRA. The workability of this mixture was good and the test specimens were consolidated easily. This is expected as the natural aggregates had favorable properties. The F.M. of the UCD sand was 2.74 which is within

recommended value ranges for a workable mixture. The UCD rock was a rounded stone which would also contribute to the workability. Secondly, as will be discussed later, the air content of this mixture would have also helped tremendously.

Mixture #6 which was the 75% recycled mixture achieved a slump of 3.0 inch (76 mm). This mixture had 75% recycled aggregates and GGBFS and 25% natural aggregates and PC. Eucon 37 was the HRWRA and a dosage of 350 ml (26.7 fl oz/cwt) was used. Procedure #2 was followed for adding the HRWRA. This mixture was easy to work with and consolidation of test specimens was done with ease.

Mixture #4 had the highest slump value of 8.25 inch (210 mm). This mixture had 25% recycled aggregates and GGBFS and 75% natural aggregates and PC. Eucon 37 was the HRWRA and a dosage of 350 ml (26.7 fl oz/cwt) was used. Procedure #2 was followed for adding the HRWRA. This mixture was very easy to work with and test specimens were consolidated with little effort.

Figure 5.1 shows the effects of the different HRWRA on the slumps of the concrete mixtures batched. What can be easily seen from this figure are the effects that recycled aggregate have on slump.

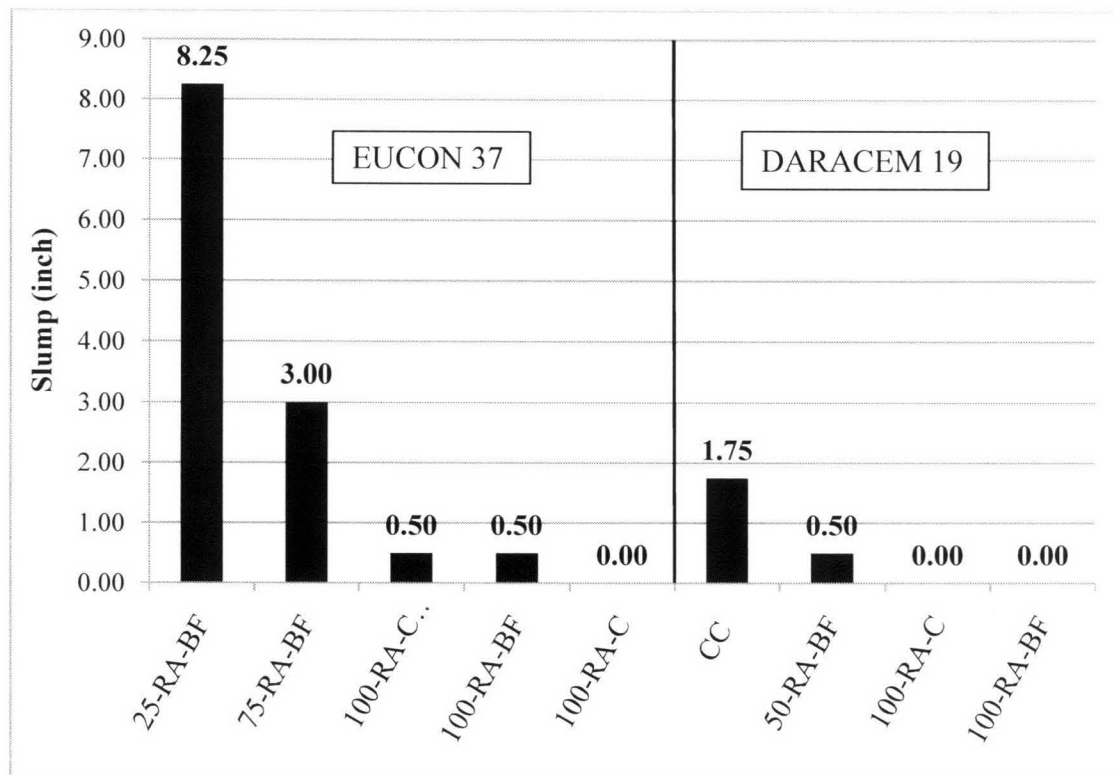


Figure 5.1 Mixture Slump and HRWRA Type Used

What can be deduced from these results is the significant effect recycled aggregates have on the workability and water demand of a concrete mixture. The RCA had an absorption capacity of 4.58% and combined with a F.M. of 4.08 for the glass and a w/c of 0.40 these results are not surprising. A significant amount of HRWRA (six times recommended dosage) was used to achieve even small slump values and after heavy dosage it became clear that HRWRA has little effect on concretes containing 100% recycled aggregates. Based on the literature review these results are consistent based on the fines content of the RCA which was above recommended guidelines. It is also notable that the HRWRA effects are not attributed to the type of cementitious used as both mixtures had similar slumps (mixtures #2 and

#3). Another noticeable difference was the effect of Eucon 37 and the Procedure #2 on the slump values. All mixtures containing Eucon 37 followed Procedure #2 and all mixtures which used DARACEM 19 followed Procedure #1. If Procedure #2 was followed with DARACEM 19 the results may have improved.

Based on these results, Mixtures #2, #3 would not be suitable for use in much more than mass concrete placements such as RCC or in placements where heavy vibration techniques could be employed. These mixtures would probably not be suitable where moderate or heavy reinforcement is expected. Mixture #5 would probably be suitable for use with reinforcement due to the ease of consolidation. Regardless of the slump value obtained on this mixture it was very easy to work with. Mixtures #1, #4 and #6 would be suitable for most applications where reinforcement is expected as these mixtures would consolidate well around the reinforcement with little vibration.

5.3.2 Air Content

The use of AEA was incorporated into all mixtures. A target air content of 6.5% was used based on CDOT Class-D mixture specifications. To achieve this, Daravair AT-60 was selected and a dosage rate of 1.5 fl oz/cwt (20 ml) was used on all mixtures. Air contents of each mixture were tested immediately after batching according to ASTM C 231 procedures. Although the dosage was measured correctly and remained consistent the air contents varied. The air contents for mixtures #2 through #6 were somewhat close to each other and the target air content. However, Mixture #1 had an air content average of 10%. This was measured four separate times with two different meters and all results were closely grouped. Although it is believed the dosage on this mixture was correct, it may have been miscalculated and would explain the excessive air content.

Mixtures #3 and #2 did not achieve the target air content having the lowest percentages of 4.6% and 5.2% respectively. This may have been caused by additional mixing time required to unsuccessfully achieve workability. As previously stated, these mixtures achieved minimal slump and while batching multiple additions HRWRA beyond the original dosage were added with intermittent mixing times between each addition. This added mixing time may have essentially deflated the concrete mix as this is a common result of over mixing. Mixture #3 was mixed for a total of 30 minutes. The air contents do not appear to be related to recycled material content at first glance. Mixture #1 which was the control mixture had the highest air content followed by Mixture #6 which was composed of 75% recycled materials. This is not consistent with mixtures #2 and #3 which had the lowest air contents and highest recycled materials contents. However, if the excessive mixing times did not occur for these two mixtures, the results may show that air content does increase with increasing recycled materials content if the control mixture is ignored assuming an incorrect dosage of AEA.

5.3.3 Unit Weight

The unit weights of each mixture were tested immediately after batching according to ASTM C 138 procedures. The design theoretical unit weights were between 133.7 and 141.9 lb/ft³ (79.3 kg/m³ and 84.2 kg/m³) for these mixtures and depended upon the amount of recycled materials and air content. The w/c was held constant for all mixtures and therefore did not play a role. The theoretical unit weights decreased with increasing recycled materials. This would be expected because the specific gravity of the GGBFS is lower than cement as well as the decreased unit weight of the RCA compared to the natural aggregates. Table 5.2 shows the theoretical and measured unit weights together with air contents for all six mixtures.

Table 5.2 Measured Unit Weights, Theoretical Unit Weights and Air Contents

Mixture Identification		Air Content (%)	Measured Unit Weight (lb/ft^3)	Theoretical Unit Weight (lb/ft^3)	Difference from Theoretical
1	CC (control)	10.0	134.8	141.9	- 7.1
2	100-RA-C	5.2	139.2	135.1	+ 4.2
3	100-RA-BF	4.6	140.8	133.7	+ 7.1
4	25-RA-BF	6.4	138.6	139.8	- 1.2
5	50-RA-BF	7.0	140.8	137.7	+ 3.1
6	75-RA-BF	7.5	134.1	135.7	- 1.6

Air content also plays a role in measured unit weights. In the design process the target air content of 6.5 % by volume is specified and accounted for when the theoretical unit weights are calculated. The air content and unit weight are inversely proportional. For example, if the measured air content was higher than 6.5 %, this gain in air content equals a decrease in paste, aggregate, etc and thus will result in a decrease in unit weight because the extra volume taken up by the weightless air. Four out of the six mixtures follow this trend of increasing or decreasing unit weight based on air content. For comparison purposes a second table was provided which shows the adjusted theoretical unit weights with the actual calculated air contents. Table 5.3 shows the adjusted theoretical unit weights.

Table 5.3 Measured Unit Weights & Air Adjusted Theoretical Unit Weights

Mixture Identification		Measured Unit Weight (<i>lb/ft³</i>)	Air Adjusted Theoretical Unit Weight (<i>lb/ft³</i>)	Difference from Adjusted Theoretical (<i>lb/ft³</i>)
1	CC (control)	134.8	136.1	- 1.3
2	100-RA-C	139.2	137.0	+ 2.2
3	100-RA-BF	140.8	136.6	+ 4.2
4	25-RA-BF	138.6	140.0	- 1.4
5	50-RA-BF	140.8	136.9	+ 3.9
6	75-RA-BF	134.1	134.1	0

Mixture #1 had a significantly lower unit weight than the theoretical but also had a much larger air content of 10%, which was 3.5 % above target. The difference in unit weight is approximately 5.2% (1.0% from adjusted). Mixture #2 had a higher unit weight which corresponds to the decreased air content of 5.2%, which is 1.3% below target. The difference in unit weights is approximately 3.0% (1.6% from adjusted). Mixture #3 had a significantly higher unit weight than theoretical and this corresponds to the lower air content of 4.6% which is 1.9% lower than target. The difference in unit weights is approximately 5.3% (3.0% from adjusted). Mixture #4 had a slightly lower unit weight but also had a slightly lower air content as well. This does not correlate with expectations but the difference in unit weights is small, only 0.85% (1.0% from adjusted). Mixture #5 had a higher unit weight than theoretical and had a higher air content as well. This does not correlate with expectations and the difference in unit weights is 2.3% (2.8% from adjusted). Mixture #6 had a slightly

lower unit weight and correlated with the increased air content of 7.5% which was 1.0% higher than target. The difference in unit weights was small, only 1.2% (0% from adjusted). Several factors may have played a role in these discrepancies. The addition of HRWRA which was not accounted for in the theoretical unit weights may have caused small differences. This would only cause very minor discrepancies however. The more probable cause of these differences is errors while performing the test. Improper consolidation or even air content measurement will cause significant differences between measured and theoretical unit weights.

5.4 Hardened Concrete Properties

Hardened concrete testing was performed on all mixtures at various ages after batching. These tests consisted of compressive strength, permeability, freeze-thaw durability and ASR expansion tests.

5.4.1 Compressive Strength

The compressive strength for all mixtures was tested according to ASTM C 39 procedures at 1, 7, 28, 56, and 90 days of age. A total of three 4 inch x 8 inch (102 mm x 203 mm) cylinders were tested at the appropriate age specified. The compressive strength is calculated by dividing the ultimate load at failure (lb) by the cross sectional area of the cylinder (in^2). This strength is then recorded as (lb/in^2).

The strength of concrete and air content are inversely proportional. The compressive strength of concrete decreases 5.0% for every 1.0% increase in air content (Mindess, 2003). In order to accurately evaluate the results of this research it was necessary to normalize the data. Since all mixtures were designed based on 6.5%

air content this value was used as a datum. Concrete strengths were then adjusted based on actual measured air content. If the air content of the concrete mixture was higher than 6.5% the strength was increased, and vice versa. Equation 5.1 shows how the data was normalized. Tables 5.3 and 5.4 summarize the average compressive and average normalized compressive strengths for all mixtures. Figure 5.2 shows failed test specimens from a 100% GGBFS mixture and a 100% PC mixture (mixtures #3 and #2) respectively.

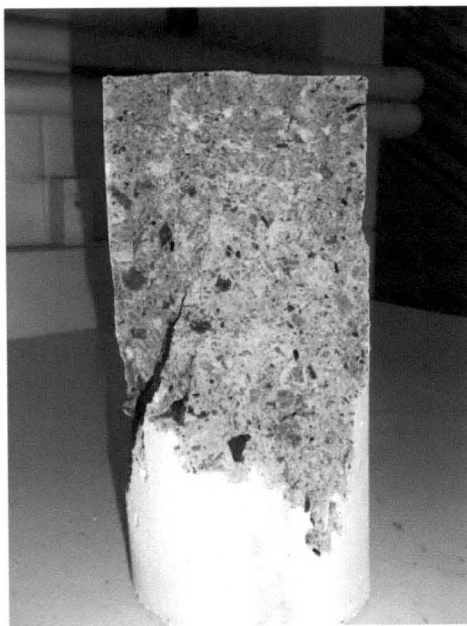
$$f'_c \times (1 - 0.05 \times 6.5\%) / (1 - 0.05 \times \text{Air Measured } \%) = f'_{c, \text{normalized}} \quad (5.1)$$

Table 5.4 Average Compressive Strengths

Mixture Identification		Compressive Strength (lb/in ²)				
		1-Day	7-Day	28-Day	56-Day	90-Day
1	Control	1974	3657	4332	4567	4824
2	100-RA-C	2410	3649	4375	4958	5540
3	100-RA-BF	773	2659	3836	4371	4820
4	25-RA-BF	2350	4248	5604	5966	6730
5	50-RA-BF	1509	5061	6674	6677	6739
6	75-RA-BF	879	4219	5148	5239	5870

Table 5.5 Average Compressive Strengths Normalized for 6.5% Air Content

Mixture Identification		Actual Air Content (%)	Compressive Strength (lb/in^2)				
			1-Day	7-Day	28-Day	56-Day	90-Day
1	Control	10.0	2665	4937	5848	6166	6513
2	100-RA-C	5.2	2113	3205	3851	4345	4869
3	100-RA-BF	4.6	677	2331	3363	3832	4226
4	25-RA-BF	6.4	2332	4216	5563	5923	6681
5	50-RA-BF	7.0	1567	5256	6931	6934	6998
6	75-RA-BF	7.5	949	4557	5560	5659	6339



**Figure 5.2 Failed Compressive Test Specimens, 100-RA-BF (Mixture #3)
& 100-RA-C (Mixture #2), Respectively**

Individual strength gain curves for each mixture are plotted and shown in Figures 5.3 through 5.7. Each individual mixture is plotted with the control mixture (CC) for comparison. These strength gain curves are all based on the normalized compressive strengths. The combined strength gain curve showing all six concrete mixtures is shown in Figure 5.8.

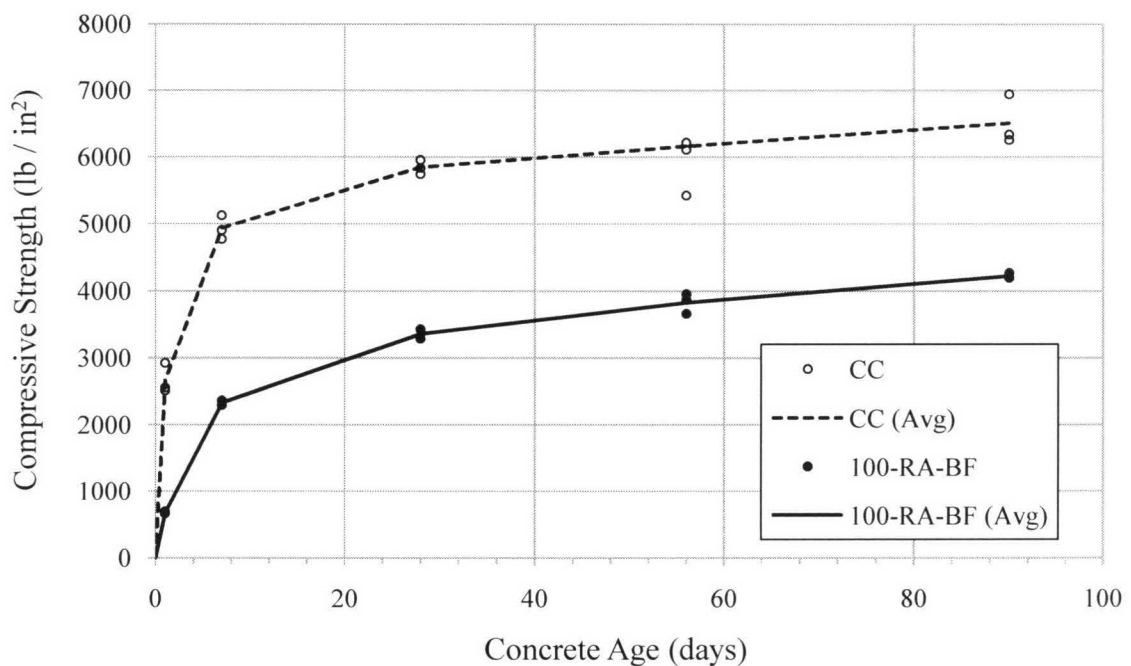


Figure 5.3 Compressive Strength Gain, 100-RA-BF (Mixture #3)

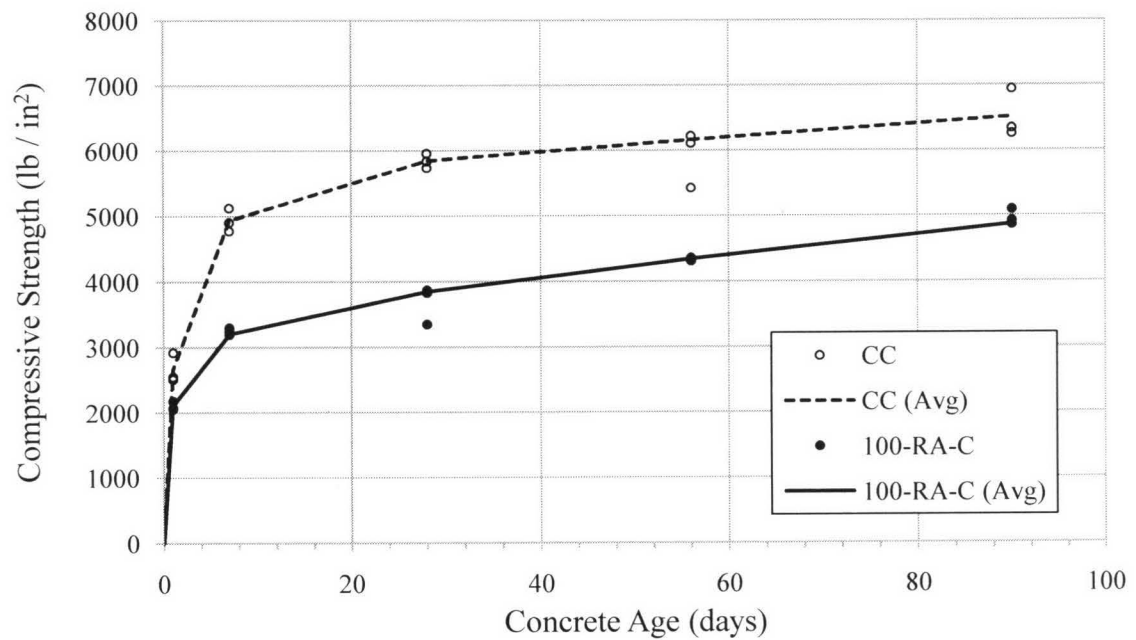


Figure 5.4 Compressive Strength Gain, 100-RA-C (Mixture #2)

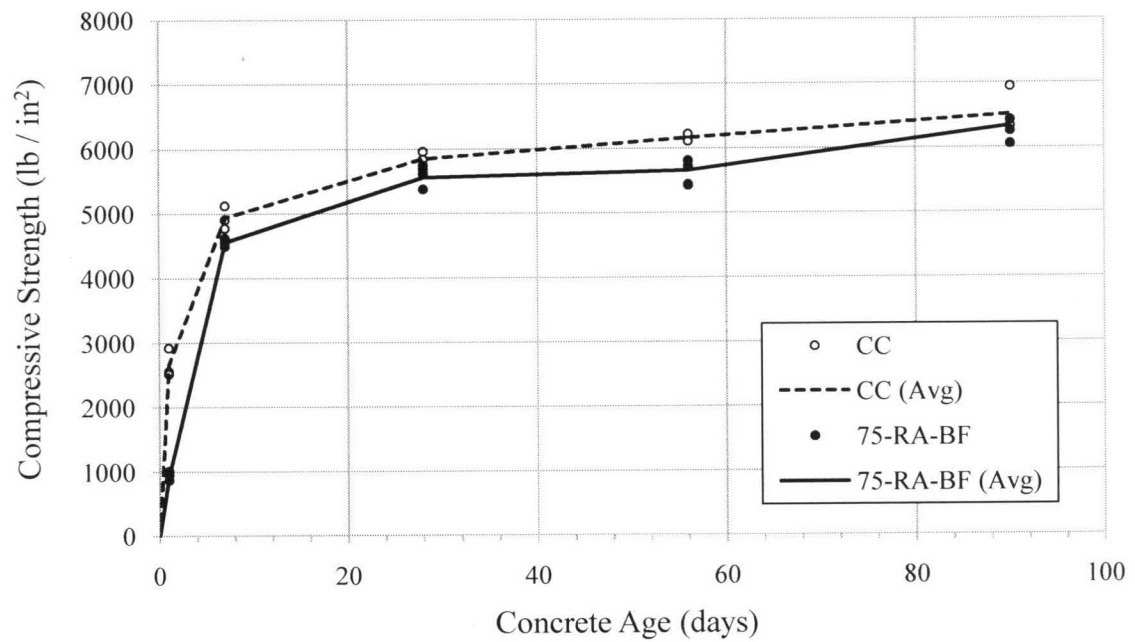


Figure 5.5 Compressive Strength Gain, 75-RA-BF (Mixture #6)

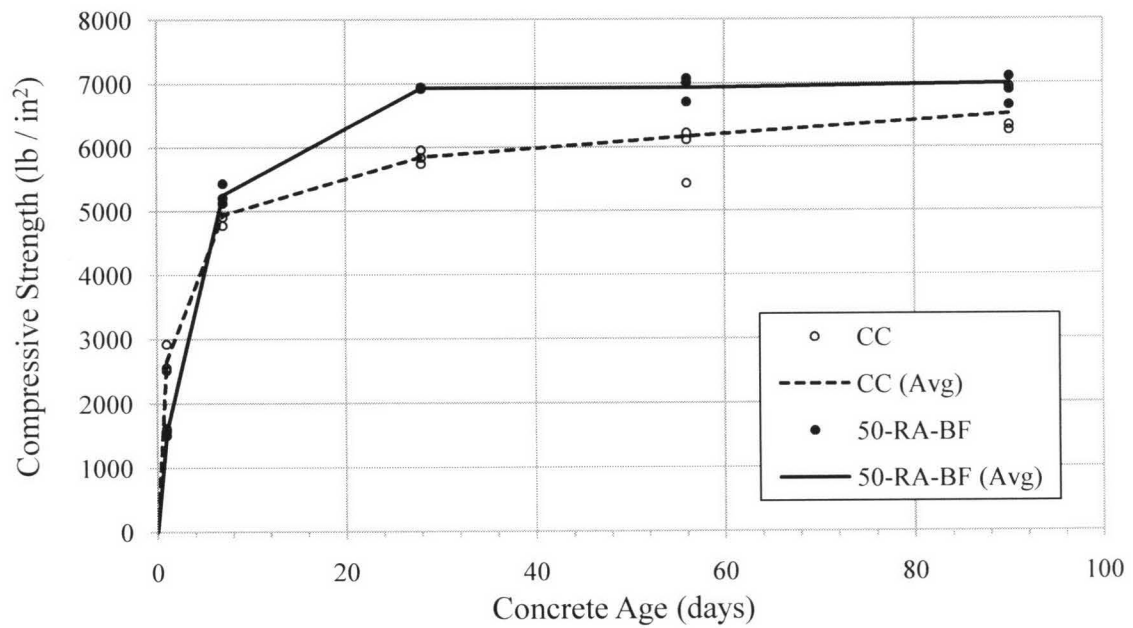


Figure 5.6 Compressive Strength Gain, 50-RA-BF (Mixture #5)

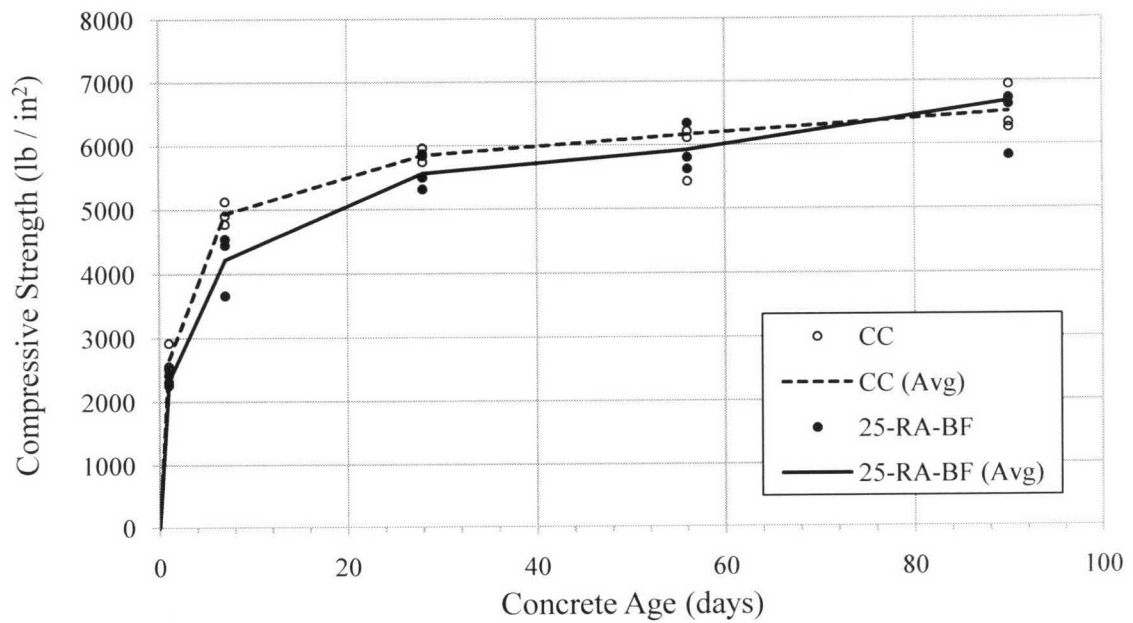
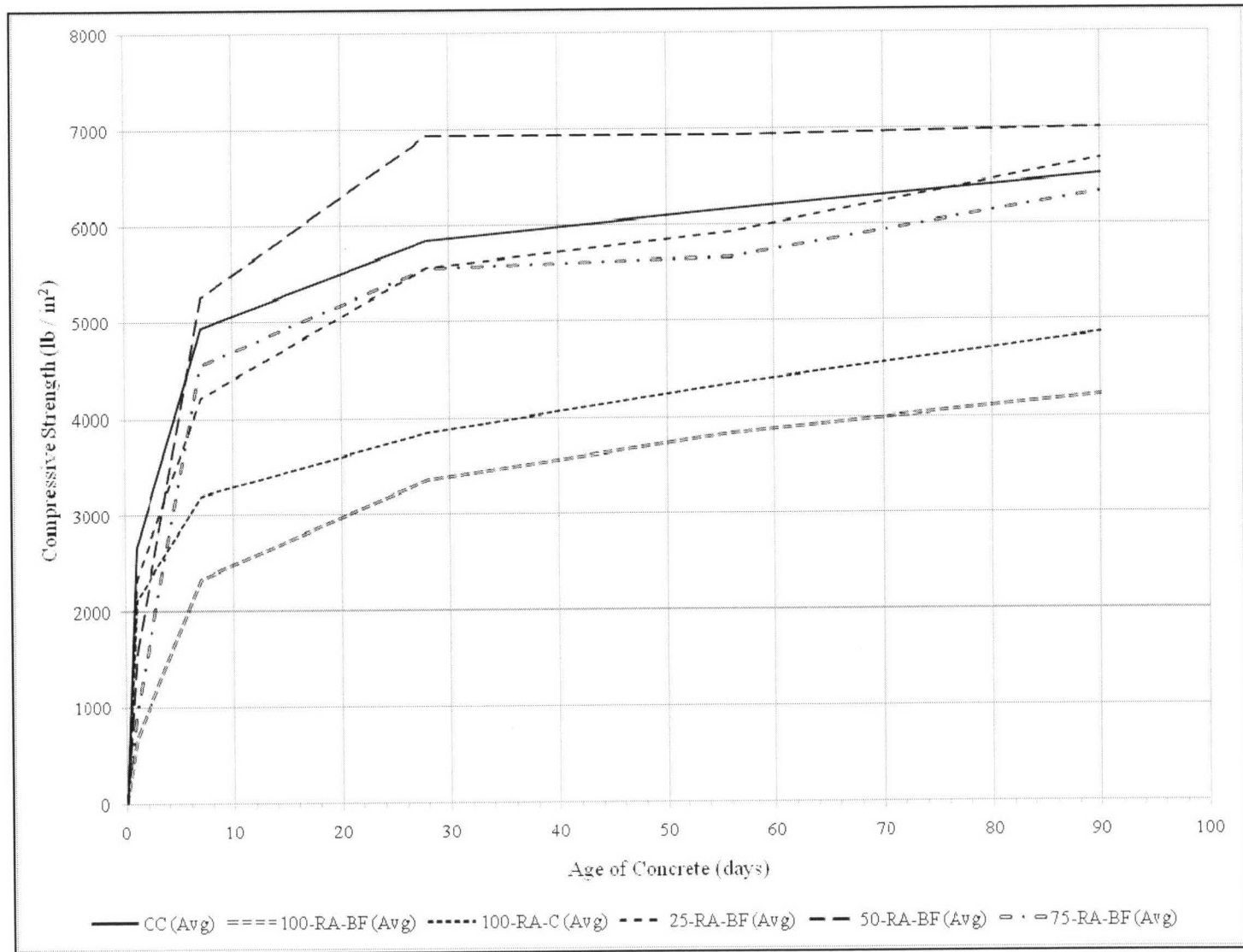


Figure 5.7 Compressive Strength Gain, 25-RA-BF (Mixture #4)

Figure 5.8 Average Normalized Compressive Strengths, All Mixtures



When comparing the results presented in Tables 5.3 and 5.4, it becomes clearly evident that air content played a large role in the compressive strengths for these concretes. The Mixture #1 (CC control) specimens did not achieve nearly the strengths that would be expected from a concrete mixture such as this one. The necessity to normalize the data based on air content should be obvious, otherwise any comparisons made between mixtures would be biased. Therefore, any further reference to concrete mixture strengths will be based on the normalized data.

Referencing Table 5.4, Mixture #1 achieved the greatest 1-day strengths when compared with the recycled mixtures. The 25% recycled mixture came close to achieving similar 1-day strengths as well. The strengths decreased for mixtures #5, #6 and #7 as recycled material content increased from 25% to 100%. Mixture #1 also had lower strengths than the control mixture. The decrease in strength for this mixture can only be attributed to the recycled aggregates as this mixture had 100% PC as the cementitious content. The lowest 1-day ultimate strength came from Mixture #3 which had no PC or natural aggregates. This is expected as the GGBFS hydrates much slower than PC. This was a pleasant surprise as it was believed that 1-day strengths would not even be possible from this mixture.

When observing the 7-day strengths, the trend of decreasing strength with increasing recycled content did not apply for this age group. Strengths increased up to 50% replacement (Mixture #5) and then decreased with further replacement. Mixture #5 had the greatest strengths of all mixtures. This strength gain of nearly 3700 psi (24.8 Mpa) in 6 days is quite impressive. What is also interesting is that this mixture had the fourth lowest strength at 1-day. This rapid increase in strength can be visualized by observing Figure 5.5. Mixture #1 maintained a steady strength gain and had the second highest 7-day strength. This was a strength gain of nearly 2300 psi (15.8 MPa) from the first day. Mixture #6 had the third highest 7-day strength, but more importantly this mixture had a very large strength gain of 3600 psi (24.8 MPa).

This strength gain is almost identical to Mixture #5 previously discussed. This would indicate that the GGBFS significantly impacts the first day strength but somewhere in between the seventh day this factor is no longer applicable. This may also be related to the PC content within mixtures #4, #5 and #6. For instance, Mixture #3 did have a decent strength gain of 1654 psi (11.4 MPa) but still had the lowest 7-day strength. This mixture must rely entirely on the GGBFS for strength as no PC is available. Although Mixture #3 had the lowest 7-day strength, the strength of 2331 psi (16.1 MPa) is a reasonable amount to strip formwork. Mixture #2 had the slowest strength gain of only 1092 psi (7.52 MPa). This is interesting because this mixture has 100% PC and the strength gain was less than the 100% GGBFS mixture (Mixture #3). When comparing Mixture #2 with Mixture #1 the effect of recycled aggregates is rather significant. The difference in strengths is over 1700 psi (11.7 MPa) for these two mixtures.

The 28-day strengths showed a drop off in strength gains for all mixtures. A similar trend of increasing strength with increasing recycled material up to 50% was observed, and a decrease thereafter. The highest strengths were produced again by the 50% recycled mixture (Mixture #5). The strength of 6931 psi (47.8 MPa) is impressive and is an increase of 1700 psi (11.7 MPa) in 21 days. What can also be seen regarding this mixture is that nearly 100% of the strength gain has occurred by 28-days as not much else occurs with strength gain hereafter. Not surprisingly the control mixture (Mixture #1) had the second highest 28-day strength. However, this mixture had the second lowest strength gain of only 900 psi (6.2 MPa). The second highest strength gain came from the 25% recycled mixture (Mixture #4) and was 1300 psi (9.0 MPa) in 21-days. Mixture #4 achieved equal strengths with the 75% recycled mixture (Mixture #6) at 28-days. Mixture #3 had a modest increase of roughly 1000 psi (6.9 MPa) but is still the lowest strength producer. However, 3300 psi (22.8 MPa) is considered a normal strength concrete and this concrete would be

able to take service loads at this point. Once again the lowest strength gain came from Mixture #2 with only 600 psi (4.14 MPa).

The progression to 56-day strengths showed another drop in strength gains for all mixtures. Again, the 50% recycled mixture (Mixture #5) had the greatest strength but this strength was the same as it was for 28-day. The second highest strength came again from the control mixture (Mixture #1). Mixture #1 had the average strength gain of all mixtures at 300 psi (2.07 MPa). The 25% recycled mixture (Mixture #4) had the third highest strength for this age group and surpassed the 75% recycled mixture (Mixture #6) for 56-day strengths. This mixture was tied with Mixture #2 for the greatest strength gain of 500 psi (3.45 MPa). Mixture #2 maintained a similar strength gain as the last age period and is still the second lowest strength producer with only 4345 psi (30 MPa) at 56-days. This mixture is only slightly higher in strength than the 100% recycled mixture (Mixture #3) which was the lowest again at only 3832 psi (26.4 MPa). Although lowest, this is a respectable strength for a normal strength class concrete.

The 90-day strength trend proved to be interesting and did not follow the usual drop in strength gains as the previous age group. Mixture #5 did not achieve any significant strength increases but still maintained the highest ultimate strength of roughly 7000 psi (48.2 MPa) at 90-days. The 25 and 75% recycled mixtures (Mixture #4 and #6) had an interesting identical strength gain of 700 psi (4.82 MPa). Both strength gains were the highest and are an increase from the 56 days of age testing group. This increase in strength gain is most significant when observing Mixture #6 which had only a 100 psi (0.7 MPa) strength gain previously. Mixture #4 surpassed the strength of the control mixture (Mixture #1) at 90-days of age with an ultimate strength of nearly 6700 psi (46.2 MPa) which is the second highest of all mixtures. Mixture #1 also had a slight increase in strength gain at 90-days of age and comes in at the third highest strength of all mixtures. Mixture #2 maintained the same strength

gain of 500 psi (3.45 MPa) and finished with the second lowest strength of nearly 4900 psi (33.8 MPa). The 100% recycled mixture (Mixture #3) had a slight decrease in strength gain from 500 to 400 psi (3.45 to 2.76 MPa). The ultimate strength at 90-days was a respectable 4200 psi (29.6 MPa).

When comparing the 28-day strength requirements for a CDOT Class-D concrete mixture, three of the recycled concretes achieved adequate strengths. The requirement is 4500 psi at 28-days of age. Mixtures #4, #5 and #6 exceeded the requirements at 28-days. The 50 and 25% recycled mixtures (Mixture #5 and #6) actually achieved the required strengths at 7-days of age. Figure 5.8 shows the 28-day strengths of all six mixtures for comparison.

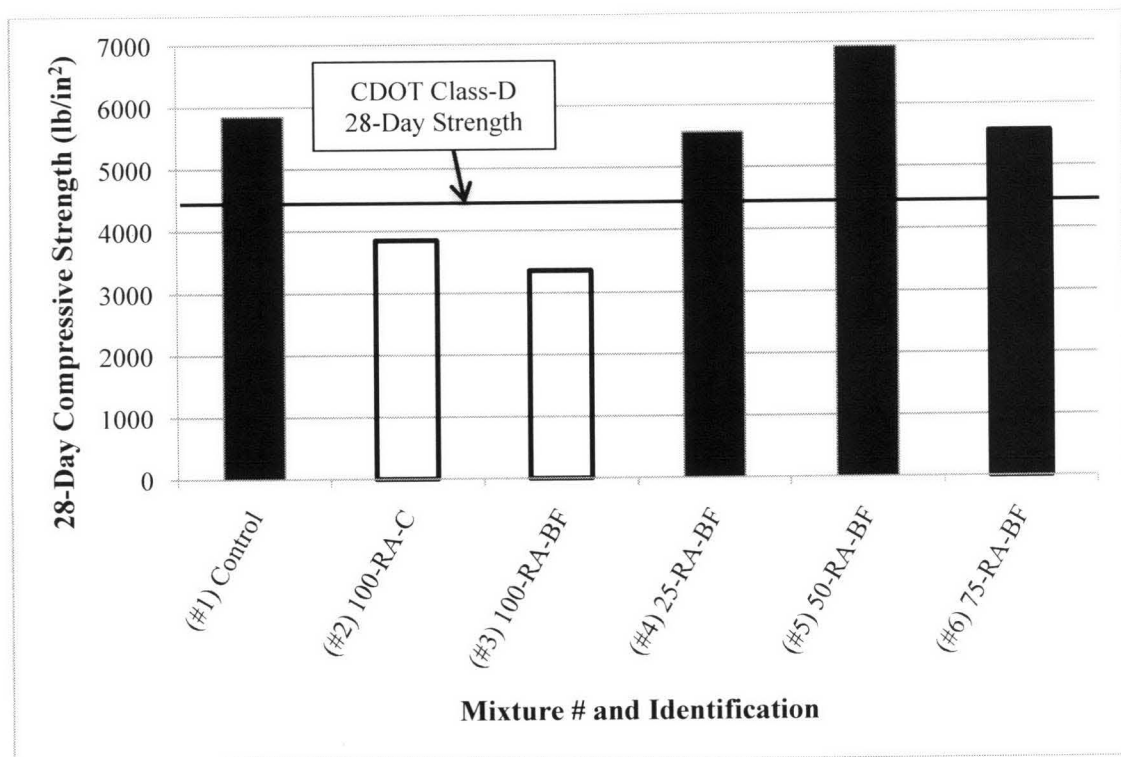


Figure 5.9 28-Day Compressive Strengths and CDOT Class-D Requirements

Even with 75% recycled materials (Mixture #6) the required strength at 28-days was achieved. However, as previously mentioned these mixtures were only loosely based on a Class-D mixture as the percentage of replacement used was beyond specifications. Also, the early age strengths for these mixtures (with the exception of Mixture #5) would also raise concern for roadway work because typically early strengths are needed to allow traffic as soon as possible. Mixture #5 not only achieved the required 28-day strength but also would be suitable for early age requirements as well. This concrete gained all its strength by 28-days and at 7-days of age this concrete would be able to take on substantial loading.

The recycled concrete mixtures overall performed well. The 50% recycled mixture (Mixture #5) outperformed the control mixture by nearly 1000 psi at 28-days and 500 psi by 90-days. This is impressive and Mixture #5 would be classified as a higher strength mixture with nearly 7000 psi at 28-days. What is interesting is that 50% recycled material content appears to be the pessimum replacement amount. Another interesting observation was the dramatic increase in strength for this mixture between 1 and 7 days of age. The 25 and 75% recycled mixtures (Mixture #4 and #6) achieved very similar strengths and strength gains after the first day. This is interesting because they are completely different mixtures with respect to recycled material contents. Both of these mixtures also achieved an identical spike of strength gain between 56 and 90-days of age. The ultimate strength of 6300 psi (43.4 MPa) for a concrete made from 75% recycled materials is quite impressive. This concrete did gain strength slowly for the first few days but then quickly caught up with its counterparts.

Mixture #3 gained the least amount of strength but at the same time is the most impressive. This strength came entirely from the GGBFS and shows that it is possible to achieve strengths without the use of PC. Although the gain is much slower an ultimate strength of 4200 psi (29.6 MPa) is a normal strength concrete. If the

strength gain was more rapid this mixture may actually be a viable alternative. However, in applications where rapid strength gain is not required (i.e. mass placements) this mixture may perform well. Although 120-day strengths were not tested for these mixtures the results would probably indicate a continued strength gain for this mixture, as well as the 75% recycled mixture (Mixture #6).

The 100% recycled aggregate mixture (Mixture #2) did not perform as well as expected. This mixture was the second lowest strength producer second only to the 100% recycled mixture (Mixture #3). This indicates that there may be something happening between the PC and the aggregate which would cause this decrease in strength. The first thought that comes to mind is ASR. It would have been beneficial to have researched intermediate replacement amounts of only recycled aggregate. Although ASR was tested and will be discussed later in the report, it would have also been interesting to leave some cylinders in the dry (i.e. not submerged) to see if ASR was a factor. This may have shown if ASR was the problem because moisture is needed to cause the ASR gel to expand. If something detrimental is occurring between the PC and recycled aggregate, this effect is obviously not occurring with a 25% GGBFS replacement (i.e. Mixture #4). It should also be noted that Mixture #2 was batched twice and the results of both batched specimens were very similar. The results presented herein are the average of these two batches.

5.4.2 Permeability

The permeability of concrete is a very important property. Susceptibility to chemical attack and corrosion of reinforcement are both directly related to a concrete's permeability. In concretes used in cold weather climates the need for a dense concrete is even more pronounced due to the constant use of chemical deicers. The testing of permeability on concrete is somewhat debatable. The most common test method used for testing a concrete's permeability is the Rapid Chloride Ion Penetrability Test (RCPT). This test method follows procedures set forth by ASTM C 1202 and was the test method used for this research program.

This test method does not actually test a concrete's permeability. Instead, this method tests a concrete's resistance to chloride ion penetration. The testing takes place for six hours and the total charge passed is used to make a relation to permeability. The reason this method is used so frequently is due to the long and complicated process of actually testing the permeability of concrete. This method is prone to problems caused by supplementary cementitious materials and the chemical transfer of electrons through the medium. Any change in the chemical composition of the cement matrix may also change the amount of charge passed. For example, if a concrete's paste volume is chemically altered due to the use of an SCM, the results may indicate a higher or lower permeability when actually this was not the case. Regardless, it has been shown to be a reliable test method and results correlate well with the standard test methods of measuring permeability. The problem with SCM's affecting concrete's permeability has mainly been contributed to silica fume. Table 5.6 shows the RCPT rating scale used to determine a concrete's permeability based on test results.

Table 5.6 ASTM C 1202 RCPT Permeability Classifications

Total Charge Passed in 6 Hours (Coulombs)	Chloride Ion Penetrability (Permeability Classification)
greater than 4,000	High
2,000 – 4000	Moderate
1,000 – 2,000	Low
100 – 1,000	Very Low
Less than 100	Negligible

All concrete mixtures developed for this research were tested according to ASTM C 1202 procedures at 28, 56 and 90 days of age. The day before each test, two concrete cylinders (4 inch x 8 inch (102 mm x 203 mm)) are removed from the water tank and a 2 inch (51 mm) specimen is cut from the top surface using diamond bladed wet saw. The specimen is then inserted into a desiccator and a vacuum is held for a total of 3 hours. The maximum vacuum held during this time was approximately 22 to 25 inches (559 to 635 mm) of mercury. After three hours water is added to the desiccator, while the vacuum is held, and the vacuum is maintained for an additional hour. The specimens are then exposed to atmosphere (the lid on the desiccator is removed) overnight for at least 18 hours. This is done to ensure the specimens are completely saturated (SSD) for the test.

On the day of testing the specimens are removed from the water and rubber gaskets are applied to both ends. Small amounts of silicone were used to ensure a tight seal between the rubber gasket and the concrete cylinder. The testing apparatus

is put together after this and two separate ion solutions are made. Each side of the concrete cylinder is enclosed in a sealed container during the test. One container is filled with sodium hydroxide (NaOH) on one side and the other container with sodium chloride (NaCl). The testing begins by applying a voltage of 60-volts between the specimen and the program Prove-It records the charge passed in Coulombs over a 6 hour period. Figure 5.9 shows the RCPT set up. Due to the amount of test containers available (4 total) only four specimens could be tested at any given time. Two specimens were tested for each mixture at the given time interval. The only exception to this was Mixture #2, which included three specimens tested for each age interval.

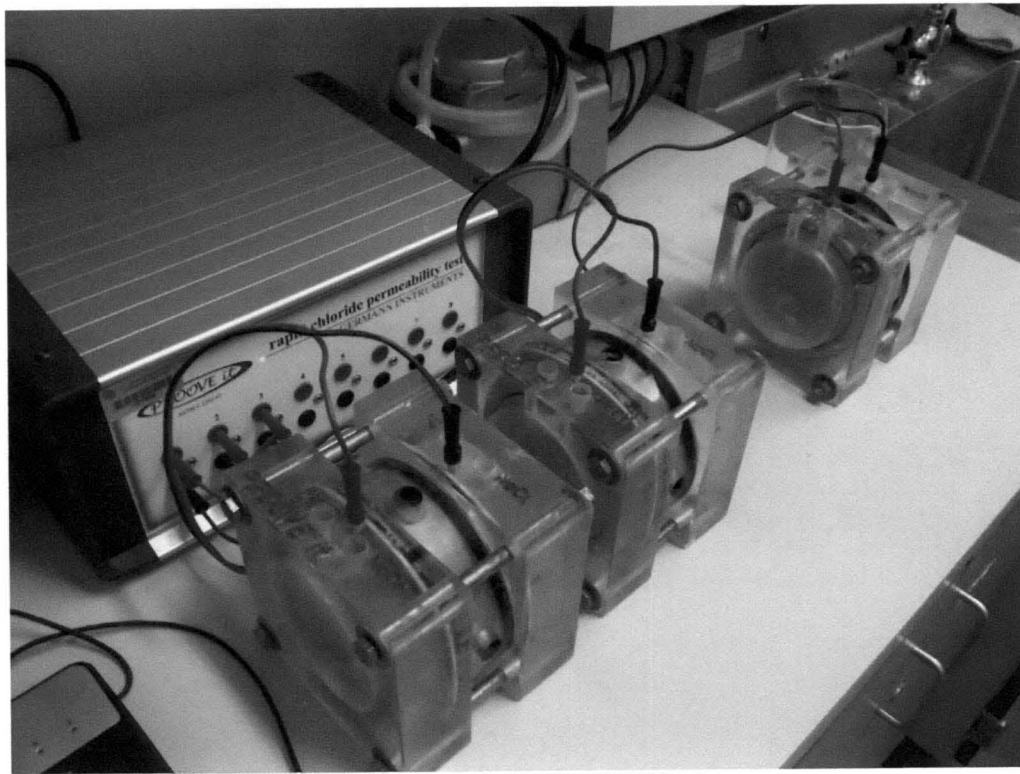


Figure 5.10 Photograph of RCPT Test Setup and Apparatus

The averaged results of the RCPT tests performed are included in Table 5.5 and Figure 5.9. These are averaged results from two specimens (Mixture #2 had three). These mixtures were somewhat designed based on a CDOT Class-D concrete mixture and there are not requirements for permeability with a Class-D. Therefore, for comparison purposes the requirements for permeability of a Class-H CDOT mixture are shown with Figure 5.9. A Class-H mixture must have an RCPT result of 2,000 Coulombs or less at 56-days of age to be approved.

Table 5.7 Results of RCPT Testing Performed at Various Ages for All Concrete Mixtures.

Mixture Identification		28 – Day Results (Coulombs)	Permeability Classification	56 – Day Results (Coulombs)	Permeability Classification	90 – Day Results (Coulombs)	Permeability Classification
1	CC (Control)	2106	Moderate	1972	Low	1969	Low
2	100-RA-C	2907	Moderate	2212	Moderate	1773	Low
3	100-RA-BF	578	Very Low	406	Very Low	321	Very Low
4	25-RA-BF	1602	Low	1326	Low	1327	Low
5	50-RA-BF	1041	Low	1003	Low	874	Very Low
6	75-RA-BF	516	Very Low	424	Very Low	403	Very Low

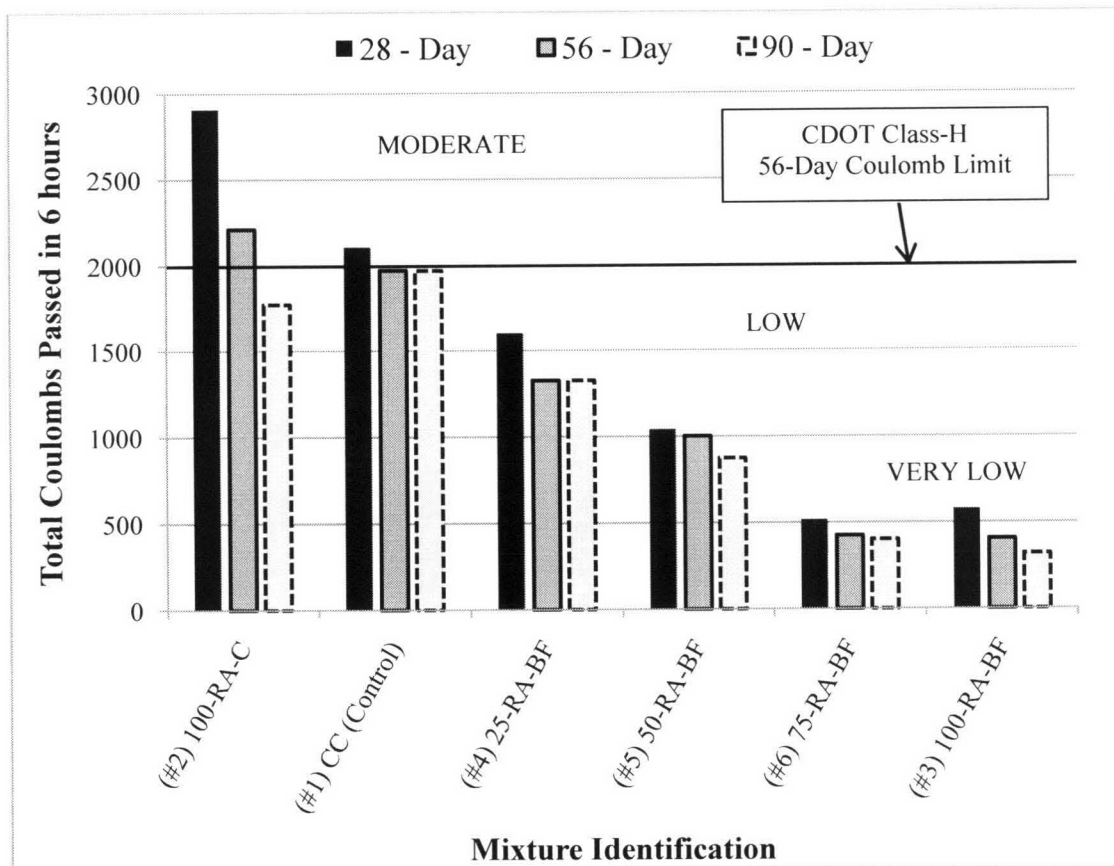


Figure 5.11 Results of RCPT Testing Performed at Various Ages for All Concrete Mixtures.

These results show that all concrete mixtures except Mixture #2 achieved the required permeability classification by 56-days of age. Mixture #2 did achieve adequate permeability by 90-days of age. What can be seen from observing these results is the dramatic impact that GGBFS has on concretes permeability. The decrease in permeability is linearly related to the amount of GGBFS contained within the mixture. This drop in permeability cannot be related to the recycled aggregates

because significantly different results were obtained from Mixture #2 as opposed to Mixture #3 which both had 100% recycled aggregates.

When observing the 28-day results, the drop in Coulombs for each increase in GGBFS content is around 500 when comparing mixtures #4, #5 and #6. Similar results were seen for the 56 and 90-day tests as well. The increase in GGBFS content from 75% to 100% doesn't appear to affect the outcome of this test by much. Both Mixture #3 and Mixture #6 achieved similar results throughout this testing. However, the ultimate lowest results did come from Mixture #3 which had 100% GGBFS. Another interesting point to be made is the performance of Mixture #2. This mixture performed the worst out of all mixtures and was composed of 100% PC as the cementitious. It was noted that consolidation concerns arose when this mixture was batched, however the very same consolidation concerns were also present for Mixture #3 which was batched at the same time. Therefore, consolidation was probably not the cause of the higher permeability.

The results of these tests are not surprising. Previous testing on the permeability of GGBFS concretes has shown similar results. The ability of GGBFS to decrease concretes permeability is well known. The decrease in permeability for Mixtures #4, #5 and #6 can be attributed to the pozzolanic reaction that takes place between the GGBFS and the calcium hydroxide (CH) released during the hydration process. This drop in permeability occurs when any pozzolanic material is used in concrete and is not limited to GGBFS. Fly ash and silica fume will also behave similar to GGBFS with respect to decreasing permeability. However, the 100% GGBFS mixture (Mixture #3) did not have any cement hydration products to allow a pozzolanic reaction to occur. As previously stated in the literature review, a change in ion concentration (OH and K) may cause these results when GGBFS is used (Shi, 2005).

5.4.3 Freeze-Thaw Durability

The freeze thaw durability of a concrete is directly related to a concrete's strength and permeability. Concrete is porous by nature and will absorb water. How much water it absorbs will depend on the permeability as well as the other factors. The more permeable a concrete is the more easily water will penetrate into the concrete matrix. Regardless of permeability, concrete will almost undoubtedly contain at least small amounts of trapped water within the matrix. When temperatures within the concrete drop to levels that allow the entrapped water to freeze, expansive forces are created due to the ice crystal formation. These expansive forces are caused by the volume change between the liquid and solid phases. Much like water penetration, ice crystal formation follows a similar principle of least resistance. Air voids within the concrete will allow ice crystal formation and thus decrease the expansive forces.

The use of concretes containing entrained air to help resist freezing and thawing cycles is well established. Due to the low air content of a normal concrete mixture (typically around 1 – 2%) air entraining admixtures (AEA) were developed to help trap and entrain more air during the mixing process. The end result is more air content within the hardened concrete matrix (typically 4 – 7 %). An in depth discussion on AEA is not within the scope of this research but the general idea has been established. Most areas where freezing and thawing cycles occur require the use of air AEA within the concrete. Although air content decreases a concrete's strength, the substantial decrease in internal forces caused by air entrainment far exceeds any concern over the reduction in strength. However, this does not imply the strength is not important. Although air helps alleviate the internal stresses, it will not eliminate them and strength will also be an important factor. To put strength in perspective, it is estimated that a concrete with a w/c of 0.36 that has fully hydrated will not need any air entrainment at all to withstand freezing and thawing cycles (PCA, 2005).

To test the ability for these research concretes to undergo freezing and thawing cycles ASTM C 666 Procedure A was chosen as the test method. Figure 5.12 shows freeze-thaw test specimens made from 100% GGBFS and 100% PC (mixtures #3 and #2) prior to testing.



**Figure 5.12 Freeze-Thaw Test Specimens, 100-RA-BF (Mixture #3)
& 100-RA-C (Mixture #2), Respectively**

For this research, two freeze-thaw beam specimens were fabricated from each concrete mixture. The specimens were 3 inch x 4 inch rectangular prisms with a length of 16 inch (75 mm x 100 mm x 405 mm). A total of 14 beams were fully tested due to the re-batching of Mixture #2. Therefore, 4 beams total were tested for Mixture #2 and two beams for each other mixture. Although ASTM C 666 states that

specimens shall be tested at 14 days of age, this is not a realistic approach when dealing with concretes containing large amounts of supplementary cementitious materials due to the slower strength gains. Therefore, it was decided to wait until 28 days of age before testing. The day testing began, the beams were removed from the water tank and weighed. After weighing the beams were tested for Static and Dynamic transverse frequencies. These frequencies were tested following ASTM C 666 procedures. First the static frequency is taken and next the dynamic. The static is only taken for comparison purposes because both should be relatively the same. These frequencies are taken prior to any freeze-thaw cycling to establish a zero point. During the freezing and thawing the frequency will decrease as ice crystal formations within the concrete break apart the micro structure and hence, the frequency will deteriorate. How much these frequencies drop over the course of the testing will help establish a durability factor (DF). Once frequencies were tested and weights determined, the beams were transported to the freeze-thaw chamber. Each beam was placed in a separate container and filled with water such that the beam was completely submerged and testing commenced.

Each week, the beams were removed from their molds during a thawing cycle and the static and dynamic frequencies were recorded. The weights of each specimen were taken immediately after removal from the water and recorded as well. After testing the beams are then placed back into the freeze-thaw chamber and testing continues. The duration of testing for these specimens was set at 300 cycles. The freeze-thaw chamber had multiple problems during this testing. The general number of cycles that occurred each day ranged from 4 to 8 cycles per day. The temperature difference during a complete cycle is roughly 40 °F. The low temperature achieved by the freeze-thaw chamber was 0 °F and the high was 40 °F (-18 °C to 4 °C). Figures 5.13 and 5.14 show the frequency testing apparatuses used.

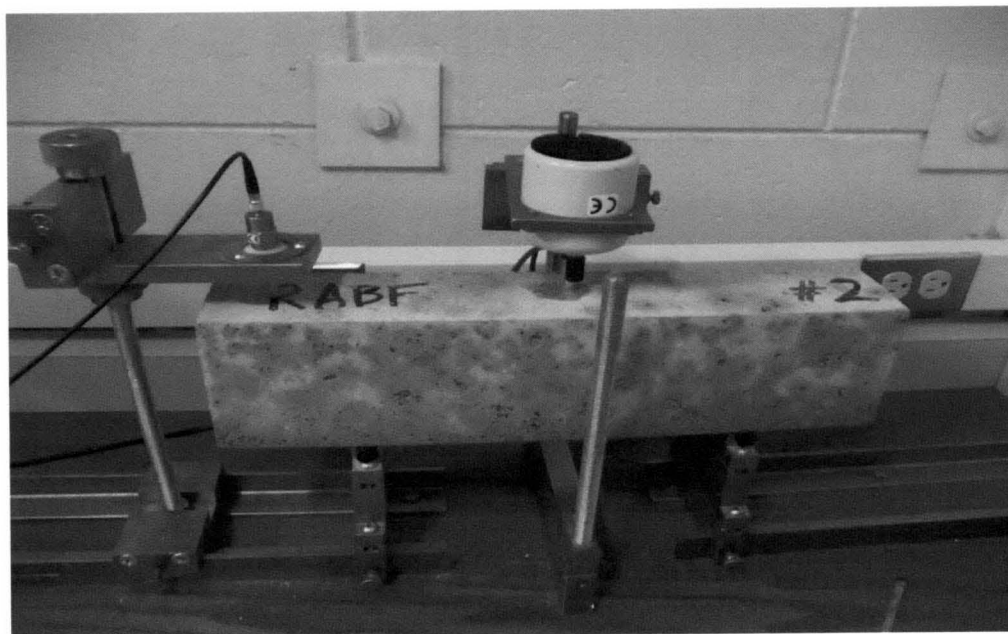


Figure 5.13 Static Frequency Test Apparatus



Figure 5.14 Dynamic Frequency Test Apparatus

The calculated dynamic frequency and the corresponding number of freeze-thaw cycles the specimens have been exposed to will be used to determine two parameters. The relative dynamic modulus of elasticity and the durability factor are calculated from these values. These two values are used to establish a relationship between the loss in frequency and the durability of a concrete. The relative dynamic modulus of elasticity (P_c) is used to calculate the durability factor (DF). P_c is a ratio of a concrete's initial frequency before testing, to the frequency determined after a certain number of cycles (n). Every week specimens are removed from the freeze-thaw machine and after frequency testing, a new P_c is determined. According to ASTM C 666 procedures, the testing should continue for at least 300 cycles of freezing and thawing or until the new P_c has dropped to 60% of the initial P_c . Equation 5.2 shows how P_c is calculated.

$$P_c = (n_f^2 / n^2) * 100 \quad (5.2)$$

Where:

n_f = fundamental transverse frequency after c cycles of freezing and thawing

n = fundamental transverse frequency before testing (0 cycles)

The relative dynamic modulus of elasticity (P_c) has been calculated for all concrete specimens and is included in the following tables along with the static and dynamic frequencies (Table 5.8 through 5.14). As previously stated, the P_c is calculated from the dynamic frequency. The static frequency is shown for comparison purposes only. If a specimen's P_c value dropped below 60%, the value is shaded in gray.

Table 5.8 Mixture #1, CC (Control), Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	1946	1934	100.0	1910	1934	100.0
34	1870	1875	94.0	1906	1875	94.0
56	1874	1895	96.0	1917	1914	98.0
96	1915	1914	98.0	1950	1934	100.0
112	1940	1914	98.0	1937	1914	98.0
168	1921	1914	98.0	1921	1914	98.0
224	1909	1914	98.0	1909	1914	98.0
320	1897	1895	96.0	1915	1904	97.0

Table 5.9 Mixture #2, 100-RA-C, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	2106	2090	100.0	2122	2090	100.0
28	2026	2031	94.5	2034	2031	94.5
77	1855	1855	78.8	1943	1953	87.3
127	1759	1738	69.2	1913	1914	83.9
152	1687	1680	64.6	1835	1816	75.5
177	1701	1660	63.1	1933	1934	85.6
202	1712	1680	64.6	1942	1934	85.6
230	1600	1582	57.3	1873	1875	80.5
258	1584	1543	54.5	1875	1875	80.5
288	1519	1523	53.1	1860	1855	78.8
314	1514	1504	51.8	1839	1855	78.8

Table 5.10 Mixture #2 (Re-Batch), 100-RA-C-2, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	2031	2051	100.0	2043	2031	100.0
25	2023	2031	98.1	1997	2012	98.1
50	1968	1992	94.4	1864	1855	83.4
75	2019	2012	96.2	1813	1797	78.3
103	1888	1895	85.3	1421	1406	47.9
131	1804	1816	78.4	1182	1133	31.1
161	1565	1582	59.5	964	938	21.3
187	1413	1406	47.0	804	781	14.8
214	1257	1211	34.9	614	625	9.5

Table 5.11 Mixture #3, 100-RA-BF, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	1995	1973	100.0	1998	1953	100.0
28	1915	1934	96.1	1880	1875	92.2
77	1701	1699	74.2	1699	1699	75.7
127	1751	1738	77.6	1725	1719	77.4
152	1727	1718	75.9	1711	1699	75.7
177	1762	1797	83.0	1672	1660	72.2
202	1774	1777	81.2	1648	1621	68.9
230	1569	1563	62.7	1427	1426	53.3
258	1573	1543	61.2	1484	1484	57.8
288	1531	1523	59.6	1406	1406	51.8
314	1570	1563	62.7	1458	1445	54.8

Table 5.12 Mixture #4, 25-RA-BF, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	2051	2070	100.0	2127	2129	100.0
49	1982	1992	92.6	2032	2031	91.0
99	1973	1973	90.8	2034	2051	92.8
124	1966	1973	90.8	2029	2031	91.0
149	1987	1992	92.6	2045	2051	92.8
174	2003	2012	94.4	2070	2070	94.6
199	1987	1973	90.8	2040	2031	91.0
227	1987	1992	92.6	2063	2051	92.8
257	2007	1992	92.6	2065	2051	92.8
283	2014	1992	92.6	2061	2031	91.0
310	2014	2012	94.4	2087	2070	94.6

Table 5.13 Mixture #5, 50-RA-BF, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	2109	2109	100.0	2135	2129	100.0
34	2082	2051	94.5	2093	2090	96.4
56	2038	2051	94.5	2077	2070	94.6
96	2113	2109	100.0	2139	2129	100.0
112	2107	2090	98.2	2154	2148	101.8
168	2090	2090	98.2	2129	2129	100.0
224	2063	2090	98.2	2055	2129	100.0
320	2067	2051	94.5	2093	2080	95.5

Table 5.14 Mixture #6, 75-RA-BF, Transverse Frequencies & P_c Values

Cycles	Specimen A			Specimen B		
	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)	STATIC (Hz)	DYNAMIC (Hz)	P _c (%)
0	2020	2051	100.0	2048	2031	100.0
49	1957	1953	90.7	1973	1973	94.3
99	1936	1934	88.9	1922	1914	88.8
124	1928	1934	88.9	1943	1914	88.8
149	1906	1914	87.1	1914	1914	88.8
174	1928	1914	87.1	1938	1934	90.6
199	1893	1893	85.2	1912	1914	88.8
227	1877	1875	83.6	1903	1895	87.0
257	1873	1855	81.9	1905	1895	87.0
283	1844	1836	80.1	1879	1875	85.2
310	1867	1855	81.9	1889	1895	87.0

The average P_c values for each concrete mixture have also been calculated. These values are useful when comparing separate mixtures. Most specimens from each individual mixture performed similar to each other and these averaged values are very representative of the actual mixture. For instance, three out of the four specimens from the 100-RA-C mixture (Mixture #2 and Mixture #2 (Re-Batch)) failed before the testing was complete. The averaged P_c values calculated from the averaged dynamic frequencies for both specimens (Specimen A and B) are included in the following tables (Tables 5.15 through 5.21).

Table 5.15 Mixture #1, CC (Control), Average Dynamic Frequencies & Corresponding P_c Values from Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	1934	100.00
34	1875	94.03
56	1904	96.99
96	1924	98.99
112	1914	97.99
168	1914	97.99
224	1914	97.99
320	1899	96.50

Table 5.16 Mixture #2, 100-RA-C, Average Dynamic Frequencies & Corresponding P_c Values from Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	2090	100.00
28	2031	94.47
77	1904	83.03
127	1826	76.36
152	1748	69.97
177	1797	73.93
202	1807	74.73
230	1729	68.41
258	1709	66.87
288	1689	65.35
314	1680	64.60

Table 5.17 Mixture #2 (Re-Batch), 100-RA-C, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	2041	100.00
25	2021	98.10
50	1924	88.85
75	1904	87.05
103	1650	65.39
131	1475	52.20
161	1260	38.10
187	1094	28.72
214	918	20.23

Table 5.18 Mixture #3, 100-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	1963	100.00
28	1904	94.12
77	1699	74.94
127	1729	77.54
152	1709	75.78
177	1729	77.55
202	1699	74.94
230	1494	57.94
258	1514	59.47
288	1465	55.69
314	1504	58.70

Table 5.19 Mixture #4, 25-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	2100	100.00
49	2012	91.80
99	2012	91.80
124	2002	90.91
149	2021	92.70
174	2041	94.50
199	2002	90.91
227	2021	92.70
257	2021	92.70
283	2012	91.80
310	2041	94.50

Table 5.20 Mixture #5, 50-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	2119	100.00
34	2070	95.44
56	2061	94.55
96	2119	100.00
112	2119	100.00
168	2109	99.08
224	2109	99.08
320	2065	95.00

Table 5.21 Mixture #6, 75-RA-BF, Average Dynamic Frequencies & Corresponding P_c Values for Both Specimens

Cycles	Average DYNAMIC (Hz)	Average P_c (%)
0	2041	100.00
49	1963	92.49
99	1924	88.85
124	1924	88.85
149	1914	87.95
174	1924	88.85
199	1904	86.98
227	1885	85.28
257	1875	84.39
283	1855	82.64
310	1875	84.39

The durability factor (DF) is calculated using a ratio of number of cycles completed at the time of test termination (N), to the number of cycles when the test was to be terminated (M). The number of cycles at test termination (N) is determined when either the P_c value drops below 60%, or the number of cycles planned for the testing has been reached (M), whichever is less. For this research it was determined to test these specimens for 300 cycles minimum, and therefore M is equal to 300. This ratio is then multiplied by the P_c at the time of test termination. Equation 5.3 shows how DF is calculated.

$$DF = (P * N) / M \quad (5.3)$$

Where:

P = relative dynamic modulus of elasticity at N cycles, (%)

N = number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less

M = specified number of cycles at which the exposure is to be terminated

The durability factors (DF) for each specimen have been calculated and are included in Table 5.22. The averaged DF for each mixture from both specimens combined has been calculated and is shown in Table 5.23. Those specimens that had DF fall below 60 are shaded in gray.

Table 5.22 Durability Factors (DF) for Each Specimen, All Mixtures

Mixture Identification		Air Content (%)	Specimen A			Specimen B		
			N (cycles)	P (%)	DF	N (cycles)	P (%)	DF
1	CC (Control)	10.0	320	96.0	102	320	97.0	103
2	100-RA-C	4.0	230	57.3	44	314	78.8	83
2	100-RA-C (Re-Batch)	5.2	161	59.5	32	103	47.9	16
3	100-RA-BF	4.6	288	59.6	57	230	53.3	41
4	25-RA-BF	6.4	310	94.4	98	310	94.6	98
5	50-RA-BF	7.0	320	94.5	101	320	95.5	102
6	75-RA-BF	7.5	310	81.9	85	310	87.0	90

Table 5.23 Average Durability Factors (DF) for Each Mixture

Mixture Identification		Air Content (%)	DF
1	CC (Control)	10.0	103
2	100-RA-C	4.0	64
2	100-RA-C (Re-Batch)	5.2	24
3	100-RA-BF	4.6	49
4	25-RA-BF	6.4	98
5	50-RA-BF	7.0	102
6	75-RA-BF	7.5	88

The mass of all specimens was taken at the time testing began and every week thereafter. Mass was taken immediately after removal (still saturated). The first measurement is taken as a datum for all other measurements to be compared with. The mass loss as a percentage of original was determined and plotted for all mixtures. This plot was developed from an average mass loss of both specimens combined (Specimen A and B). All specimens behaved very similar with regards to mass loss, therefore there is not any bias produced from averaging. Figure 5.15 shows the averaged mass loss for all mixtures. Most mixtures exhibited a mass gain during testing. This occurs due to the micro cracking caused within the concrete matrix. As cracking continues to increase over the course of testing, these cracks are then filled with water and the mass of the specimen slightly increases.

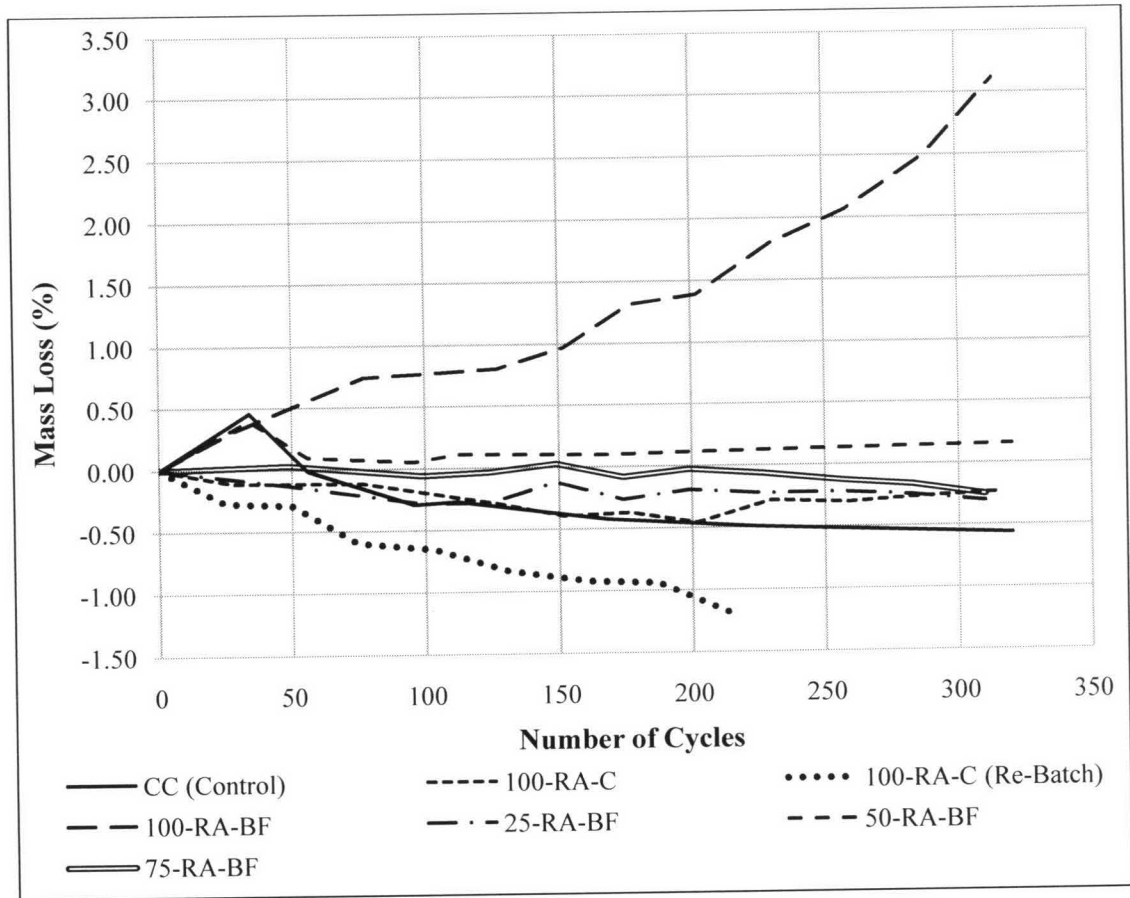


Figure 5.15 Average Mass Losses vs. Number of Cycles for All Mixtures

The results show that the recycled materials content may have an effect on the durability of a concrete mixture. When observing these tables it can be seen that the recycled materials do not have an effect when used at replacement levels up to 50%. However, at replacement levels in excess of 50% the durability factors begin to drop. What can also be seen is the difference in durability with regards to air content. Mixtures #1, 2, 4, 5 and 6 all had air content at or above the target air content of 6.5%. All of these mixtures also had durability factors above 80. A concrete with a

DF of 60 or more is considered to be able to perform well in actual conditions where freezing and thawing occurs. This is an arbitrary opinion though as a DF above 60 is no guarantee that a concrete will behave well in actual freezing and thawing conditions (Mindess, 2003). Regardless, this method is proven and is a good means of comparing similar concretes. According to ACI 318-08, the required air content for these mixtures with NMAS of $\frac{3}{4}$ inch (19 mm) should be 6% and 5% for severe and moderate exposure climates respectively. The conditions within the freeze-thaw chamber are no doubt extreme. Secondly, ACI-318-02 also recommends that a concrete strength be 4500 psi (31 MPa) for extreme exposure climates. Only three out of the six mixtures achieved these strengths by the time testing began.

Mixture #1, CC (Control) performed well as expected. The air content for this mixture was much higher than other mixtures at 10%. As previously discussed, this air content was measured several times and the results were all similar. It was believed a mistake in dosage must have occurred during batching. This extra air content would have certainly helped out the durability. The strength of this mixture even with 10% air content was 4300 psi (29.6 MPa) at 28-days. Although below guidelines, the strength was close to recommendations. There was a very small amount of pitting observed on both specimens on the surface. The general trend for this mixture was a mass gain during testing. The initial spike seen in the mass loss plot was probably caused by allowing the specimen to dry out before taking a reading. Concrete rapidly loses moisture when removed from water and if a measurement is not taken immediately, these discrepancies will occur.

Mixture #2, 100-RA-C and 100-RA-C (Re-Batch), did not perform well during testing. With regards to P_c , these specimens deteriorated the most rapidly when compared with all other mixtures. Only one specimen out of four had a P_c value above 60% by the end of testing. This specimen did show a similar trend of decreasing P_c and would have probably failed if a few more cycles were performed.

The average DF of 64 for 100-RA-C is deceiving because three out of four specimens had DF values below 50. The three specimens that did fail showed signs of deterioration early on during testing. Fractures running the length of the specimen were visible and grew in size as testing continued. The air content of both these mixtures was below 6.0% and the average strength at time of testing was 4375 psi (30.1 MPa). The strength was probably not the contributing factor as the control mixture (mixture #1) had similar strengths at 28-days. These mixtures could have been experiencing ASR expansion from the waste glass, and the freezing and thawing cycles may have exacerbated this mechanism. This may explain the aggressive mass gain exhibited by these specimens. The fractures within the concrete caused by deterioration may have allowed more water than usual to penetrate into areas that were formally occupied by paste. This concrete mixture would not be suitable for severe or moderate exposure climates.

Mixture #3, 100-RA-BF, did not perform well and showed the most rapid mass loss of any other specimens. With regards to DF, both specimens had values below 60. The P_c values for these specimens dropped rapidly after only 28 cycles. Both specimens failed prior to test completion. Specimen A failed at 288 cycles while Specimen B failed much earlier after only 230 cycles. The mass loss observed for these specimens was the most alarming. After 28 cycles these specimens began to exhibit surface scaling. During the course of the testing this surface scaling quickly changed to spalling with larger aggregate loss. The corners deteriorated and aggregate began to dislodge from the specimens. By the end of the test these specimens showed an average mass loss of 3.0%. The air content of these specimens was only 4.6%, and this would have contributed to the rapid deterioration. Secondly, the strength of this mixture was only 3836 psi (26.4 MPa) at 28-days.

Mixture #4, 25-RA-BF, performed well during testing and had DF values above 90 for both specimens after 310 cycles. These specimens showed no

deterioration during testing and actually showed a general mass gain throughout. The air content of this mixture was 6.4% and was above guidelines. Combined with a 28-day strength of 5600 psi (38.6 MPa) these results are not surprising.

Mixture #5, 50-RA-BF, performed very well during testing and had DF values of 95 after 320 cycles. These specimens showed small amounts of surface scaling after completion but this was a limited amount and generally occurred around the corners of the specimens. These specimens did not show any mass gain like most other specimens. The percentage of mass loss was very minor however and related to the surface scaling. The lack of mass gain is interesting because all specimens that performed well also showed this trend. This mixture gained all its strength by 28-days of age and this may be the cause. The mass gain observed with most specimens is believed to be associated with micro cracking within the matrix. Since the strength of this mixture was 1000 psi (6.9 MPa) greater than the second highest strength mixture at 28-days, and had an air content of 7.0%, these specimens may have been able to resist this much of this micro cracking.

Mixture #6, 75-RA-BF, performed well during testing and had DF values of 85 and 90 after 310 cycles. This mixture performed well even with a 75% replacement level with recycled materials. These specimens exhibited minor scaling over the course of testing and this was mostly limited to the bottoms of the specimens. This scaling became progressively worse as testing continued. These specimens showed a general mass gain similar to other mixtures. The air content of this mixture was 7.5% and the 28-day strength was 5150 psi (35.5 MPa) which are both above recommendations.

A final interesting observation can be made when comparing the 25, 50 and 75% recycled mixtures (mixtures 4, 5 and 6). These mixtures all had strengths greater than the control mixture at 28-days but had between 3.5% and 2.5% lower air

contents. Although the air contents of these mixtures were sure to have caused the decent results, the strength differences between these mixtures and control, combined with the lower DF results, raise questions as to the quality of strength produced by high content GGBFS mixtures. For instance, mixture #6 had a high air content (7.5%) and strength in excess of the control, but the DF values were 13 to 18 points lower. Although the difference in results may have been caused by the difference in air contents, a secondary cause may be present. It has been postulated that the strength of GGBFS is more C_2S controlled rather than C_3S which correlates with later age, rather than early age strength gains. Whether the strength or structure of the cementitious paste formed by hydration of GGBFS is as durable as pure cement paste may be argued based on these results.

5.4.4 Alkali-Silica Reactivity (ASR)

Due to the concerns related to the use of waste glass as aggregate, several ASR tests were performed. As previously mentioned in the literature review, the use of waste glass in concrete is almost certain to cause ASR expansion. The extent of this expansion however depends on the aggregate size and gradation, as well as the color. Since mixed colored glass was used in these research concrete mixtures as well as GGBFS, it was decided to investigate whether or not ASR would pose a problem. To fully investigate this potential problem it was decided to investigate three different cementitious combinations. These mixtures were designed and batched separately from the previous six design mixtures according to ASTM C 1567 procedures.

One mixture would contain 100% waste glass as aggregate and 100% PC as cementitious materials. The next mixture would have 100% waste glass as aggregate with 50% GGBFS and PC as the cementitious materials. The third mixture would be composed of the same 100% waste glass as aggregate but contain 100% GGBFS as

cementitious materials. These three mixtures were designed to investigate whether the GGBFS would mitigate the ASR expansion and if so, help capture the replacement amount needed. Results would show whether 50% GGBFS is enough to mitigate the ASR and whether or not 100% GGBFS causes an ASR expansion. Since GGBFS contains alkalis and is an alkali activated material, the potential for alkali-silica reactivity is probable. Although there may be potential for the RCA to cause ASR expansions this was not investigated because of the obvious problem associated with the crushed waste glass. The waste glass was fully washed prior to testing and graded to match the percentage mass requirements of ASTM C 1567 procedures. Table 5.5 shows the three mixture proportions investigated. The only difference between these mixtures is the type and amount of cementitious material.

Table 5.24 Mixture Proportions for Testing ASR Potential Based on ASTM C 1567 Procedures.

Material		Mixture Identification		
		1	2	3
Cement (<i>Type I-II</i>)	(grams)	440	220	0
GGBFS (<i>Grade 120</i>)	(grams)	0	220	440
Waste Glass (retained on No. 8)	(grams)	99	99	99
Waste Glass (retained on No. 16)	(grams)	247.5	247.5	247.5
Waste Glass (retained on No. 30)	(grams)	247.5	247.5	247.5
Waste Glass (retained on No. 50)	(grams)	247.5	247.5	247.5
Waste Glass (retained on No. 100)	(grams)	148.5	148.5	148.5
Water	(grams)	207	207	207
W/C	(ratio)	0.47	0.47	0.47

To achieve these different mass proportions of waste glass, multiple gradations were performed. The gradations separated the glass into fractions retained on the various sieves. The waste glass was then washed thoroughly and placed in the oven overnight to dry. All fractions of waste glass were kept separated until the time of batching when each material was weighed out and placed into a Hobart mixer for mixing. The mixing procedure followed ASTM C 205 procedures. The batching of each mixture produces enough mortar to make three test specimens. These specimens are 1 inch (25 mm) rectangular prisms measuring 10 inch (255 mm) in length. After 24 hours, the specimens are stripped from the molds and an initial reading is taken on the lengths. Figures 5.16 and 5.17 show the preparation of test specimens, and the actual testing of a specimen, respectively.

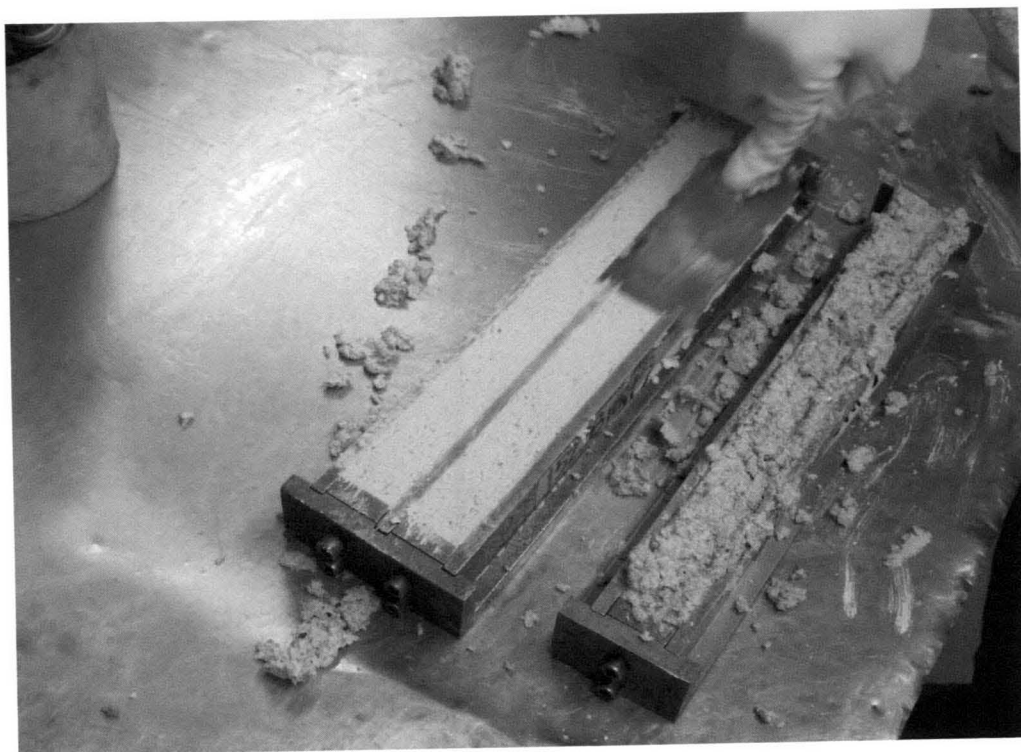


Figure 5.16 Preparing ASR Test Specimens, (100% GGBFS Mixture)



Figure 5.17 Measuring Length of an ASR Test Specimen

Because mixture #3 was composed of 100% GGBFS as cementitious, it was decided to allow these specimens to cure for an additional 24 hours prior to testing. Therefore, mixture #3 cured for a total of 48 hours (as opposed to the ASTM recommended 24 hours). After initial curing is complete, the specimens are then placed in a water bath of 80.0 °C (176 °F) for 24 hours. A new zero reading is made after this 24 hours (total of 48 hours) and the specimens are then transported to a NaOH solution for the remainder of testing. This NaOH bath is also held at a constant 80 °C (176 °F). Every three to four days, these specimens were measured longitudinally for a total of 14 days and the results of these tests are shown in Figures 5.18, 5.19 and 5.20. These graphs display the average (3 specimens) percentage of longitudinal expansion from the zero reading taken on the third day (first day of

testing, prior to submergence). According to ASTM C 1567, the potential for deleterious expansion is also displayed. If a specimen expands less than 0.10%, it is believed to be an acceptable mixture and will have a low risk of deleterious expansion. If a specimen expands more than 0.10%, the risk of deleterious expansion in the mixture exists. If a specimen expands more than 0.20%, there is a high risk of deleterious expansion in the mixture.

Copies of the actual data log sheets are included in Appendix B. The preparation of the glass was performed at the University of Colorado Denver, Materials Testing and Research Laboratory, Room 1805. The batching and testing of the specimens was performed at CTL Thompson located in Denver, Colorado. All testing of the specimens was performed by the CTL Thompson staff.

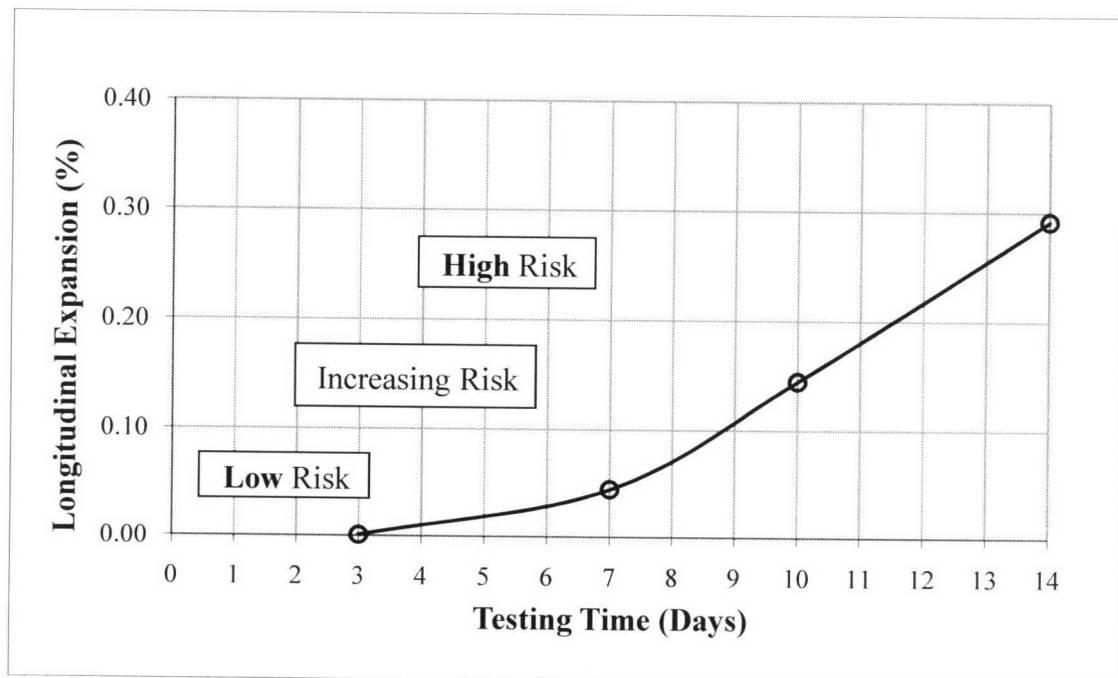


Figure 5.18 Mixture #1 (100% PC) 14-Day Test Results, ASTM C 1567

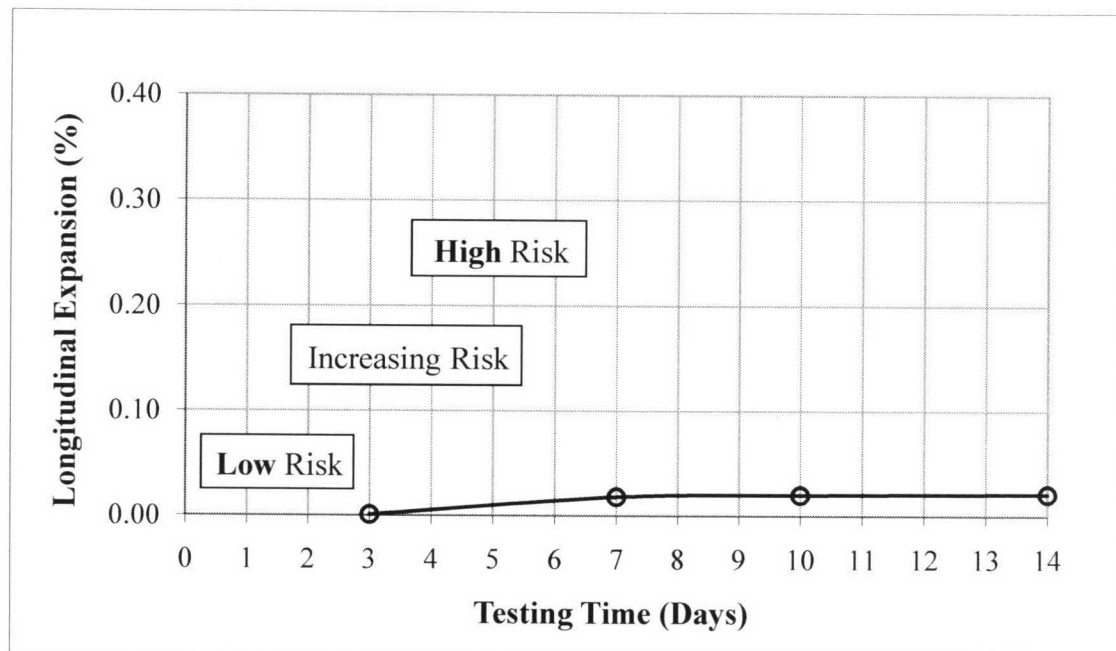


Figure 5.19 Mixture #2 (50% PC & GGBFS) 14-Day Expansions, ASTM C 1567

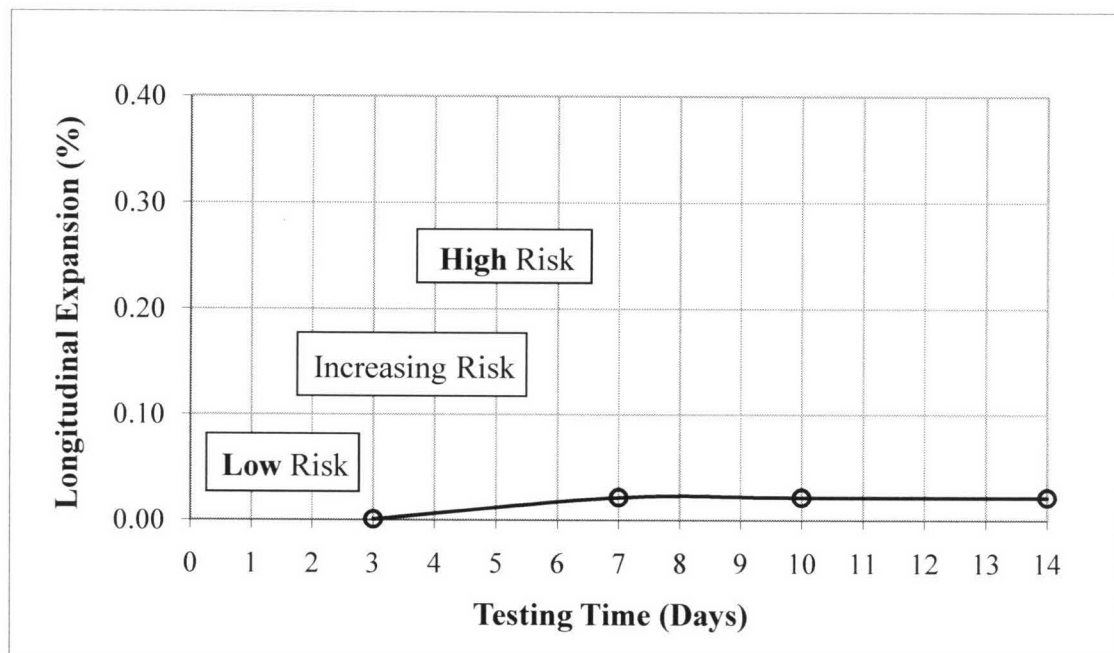


Figure 5.20 Mixture #3 (100% GGBFS) 14-Day Expansions, ASTM C 1567

These results indicate that the waste glass aggregate used in the research concretes does have potential to react with the alkalis in the cementitious paste. Mixture #1 shows that the potential for ASR expansion is high, and this combination would not be a suitable choice. This mixture had an average expansion of 0.239% (high of 0.399%, low of 0.238%). The use of 100% PC with 100% waste glass as an aggregate would not be reasonable choice based on these results.

Mixture #2 performed well. This combination with 50% GGBFS shows little risk of deleterious expansion. This mixture had an average expansion of only 0.022% (high of 0.028%, low of 0.018%). The use of 50% GGBFS as cementitious material is suitable based on these results. Whether or not a lower replacement level would have been sufficient in reducing the ASR expansion is unknown. It may be possible that replacement less than 50% could have achieved similar results. However, based on previous studies, replacement amounts in excess of 35% were required to decrease expansions below 0.10% when reactive aggregates were used in similar tests (Detwiler, 2003).

Mixture #3 also performed well. The use of 100% GGBFS as cementitious proved to be an expectable combination with the waste glass. These results are almost identical to mixture #2 and show little risk of deleterious expansion. Although this mixture was cured for an additional 24 hours, the results are still valid. This mixture had an average expansion of 0.022% (high of .030%, low of 0.018%). The use of 100% GGBFS as cementitious material in combination with waste glass is suitable based on these results.

These results are interesting in that both mixtures #2 and #3 performed equally well. The results were essentially identical. The use of GGBFS at replacements beyond 50% would therefore appear to be useless, from an ASR mitigation standpoint. Since GGBFS is alkali activated, there actually may be two

mechanisms occurring in mixture #2 and #3. There are studies on the use of alkalis (added to accelerate the hydration) in a GGBFS concrete, but how much of the concentration is used during hydration is unknown. According to ASTM C 989 and other studies, the use of GGBFS to mitigate ASR potential has been attributed to the dilution of the alkalis within the cementitious paste. This is similar to the use of other SCM's such as fly ash. However, the GGBFS may also be using the alkalis as well which would also reduce the concentration. What percentage of mitigation is achieved from either the GGBFS using the alkalis, or diluting the concentration is unknown. However, this test method does not limit the amount of alkalis and always has an abundance available for reactions to take place.

The results of this test should be used with caution. This method tests for potential reactivity between aggregates and cementitious materials, and not actual expected field conditions. For instance, comparison of mixture #2 which was made from 50% GGBFS with the 50% recycled concrete cannot be made. The 50% recycled concrete had only 50% waste glass (not 100% waste glass) and RCA as coarse aggregate. However, this test method is well accepted and can be used to give a "worst case" scenario. The 50% recycled concrete, based on these results, would probably perform well because it only has 50% waste glass as fine aggregate but has 50% GGBFS as cementitious. A decent comparison can be made with regards to the 100% recycled concrete and mixture #3. Although the 100% recycled concrete had RCA as coarse aggregate, it also had 100% waste glass as fine aggregate. Based on these results, the 100% recycled concrete would not experience deleterious expansions caused by ASR. These results may also confirm previous beliefs that the performance of the 100% recycled aggregate concrete (mixture #2 and mixture #2 re-batch) may have been attributed to ASR expansion within the concrete

6. Conclusions and Recommendations

This research study was designed to test the effects of recycled materials, in varying amounts, on the fresh and hardened properties of concrete. These tests were performed according to ASTM testing standards and when deviated from, notation was made. The purpose of this research was to determine whether these recycled materials would have negative or beneficial effects on a concrete. If these effects were detrimental, at what point they become detrimental was also of importance. The results were discussed in length in Chapter 5 of this report. The summarization of these results as well as recommendations for future studies is the basis of this chapter.

6.1 Summary

The results of this research showed that recycled materials can be incorporated into a concrete without detrimental effects. The author believes that the term “beneficial” should not be confused with “increase”. When incorporating recycled materials into a concrete mixture the benefit is immediately achieved, provided no significant detrimental effects are realized from this incorporation. When replacing natural virgin aggregates with materials that would otherwise end up in a landfill, the benefit is obvious. The use of these recycled aggregates was never believed or anticipated to “increase” a concrete’s strength or durability, rather an investigation was made into whether these materials would significantly decrease these properties when compared to a normal concrete mixture. The same principle applies to the replacement of PC with GGBFS. Ultimately the “beneficial” use of recycled materials is realized when these recycled concrete mixtures achieve similar properties when compared with a normal concrete. The primary findings of this research are summarized in the following bullets:

- Replacement levels up to 50% with recycled materials were determined to be non-detrimental to a concrete mixture with regards to hardened properties. Replacement levels up to 50% recycled materials actually enhanced concrete properties. A replacement of 50% was determined to be an optimum replacement level. A reduction in quality began to manifest at 75% and was fully visible at 100% replacement.
- Replacement of natural virgin aggregates with RCA and crushed waste glass puts a heavy burden on the workability of a concrete mixture. The extremely high absorption capacity of the RCA coupled with the harshness of the waste glass decreases concrete workability. The use of HRWRA to achieve a workable mixture is essential when using recycled aggregates. Even a small amount of virgin natural aggregates (25%) will greatly improve workability and the effects of HRWRA.
- The use of waste glass aggregates without the inclusion of GGBFS has detrimental effects on the hardened properties of a concrete. This detrimental effect is believed to be caused by the ASR expansions determined to exist between mixtures with 100% PC as cementitious and waste glass. A mixture with 100% PC and waste glass as aggregate was shown to have very high expansions when tested for ASR potential. The use of 50% GGBFS as cementitious was found to mitigate these expansions to a negligible level.
- The use of AEA was successful and was not believed to be affected by the use of recycled materials. However, increased mixing times with the use of 100% recycled materials will be detrimental on the air entrainment and must be considered.

- A replacement amount of 50% recycled materials was determined to be beneficial to a concrete mixture. A concrete composed of 50% recycled materials achieved significantly greater strengths than a control mixture. An ultimate strength of nearly 7000 psi (48.3 MPa) at 28-days of age was achieved. The permeability of this mixture was also much lower than the control concrete. The freeze-thaw durability of this concrete was substantial and not affected by the recycled materials. Based on hardened properties, this concrete would be suitable for any applications where the control concrete was specified, if not more due to the lower permeability and higher strength.
- A replacement amount of 25% recycled materials was determined to be beneficial to a concrete mixture. A concrete composed of 25% recycled materials achieved slightly lower early age strengths and greater later age strengths than a control mixture. An ultimate strength of nearly 6700 psi (46.2 MPa) at 90-days of age was achieved. The permeability and freeze-thaw durability of this concrete was equal to a control mixture. This concrete would be suitable for any applications the control mixture would be specified.
- A replacement amount of 75% recycled materials was determined to be non-detrimental to a concrete mixture. A concrete composed of 75% recycled materials achieved slightly lower early age and later age strengths than a control mixture. An ultimate strength of nearly 6350 psi (43.8 MPa) was achieved at 90-days of age. The permeability of this concrete was significantly lower than the control mixture. The freeze-thaw durability of this mixture was only slightly lower than that of a control and was still considered satisfactory (DF above 80). This concrete would be suitable for use in many applications. The strength of this concrete is adequate for many uses, especially when combined with the very low permeability and adequate freeze-thaw durability.

- A replacement amount of 100% recycled materials was determined to have detrimental effects on a concrete when compared with a control mixture. The compressive strength of a concrete composed of only GGBFS is significantly lower at all age groups. An ultimate strength of 4200 psi (29.0 MPa) was achieved at 90-days of age. The permeability of this concrete was significantly lower than the control mixture. The freeze-thaw durability of this concrete is very low. Although decreased qualities were observed with a replacement of 100% recycled materials, this concrete would be suitable for certain applications. This concrete achieved a normal strength classification at 90-days of age and if slow strength gain is not a concern, as well as climate, this concrete may be a suitable candidate. Coupled with the immense benefit of using a concrete composed of entirely recycled materials, there is most certainly many applications where this concrete could be used successfully.

This research has shown that the use of recycled materials is beneficial to not only the environment, but also on the properties of a concrete. Although these recycled materials did have an effect on the fresh properties of a concrete, primarily workability, these obstacles could easily be overcome with effort and planning. Substantial amounts of recycled materials were used with no significant detrimental effects. This research has shown that a concrete does not need PC to develop strength. GGBFS is a viable alternative to PC and should be considered when selecting cementitious content. Since aggregates are by and large a filler material, the use in concrete should be considered. However, the use of recycled aggregates should be considered with caution. This research has shown that the use of recycled aggregates alone may pose problems without the use of GGBFS. Deleterious expansions caused by ASR between the waste glass and PC were shown to exist when no GGBFS was

used. These expansions were insignificant when replacements of 50% GGBFS were used.

6.2 Recommendations for Future Studies

The first batching of this research involved two concrete mixtures which were composed of 100% recycled aggregates. The extreme water demand that the RCA has on a concrete mixture was not fully realized at the time these mixtures were batched, and re-batched. Two complete concrete mixtures were lost due to the workability and inadequate consolidation. Unfortunately the literature review was performed after much of the testing was performed and if this was performed prior to batching, results may have been more favorable. The literature review on RCA and waste glass showed that these two aggregates put a heavy burden on a concrete, especially the RCA. Based on this research and experience, the RCA should be in an SSD condition at the time of batching. This was shown to have a great effect on the slumps of concretes made with RCA as aggregate. Even if the RCA is beyond saturated and moisture contents are taken accurately, this will definitely benefit the workability.

When batching mixtures with or without recycled materials, the method of splitting up the water (50/50) and including the HRWRA in one amount, with the AEA in the other should be considered. This method proved to be much more successful than adding the HRWRA directly in these mixtures and would undoubtedly work well with other mixtures as well. This procedure was defined as Procedure #2 in Chapters 4 and 5.

The fines content of the RCA should be considered when designing mixtures. If possible, these fines should be removed from the aggregate prior to batching. The literature review showed that these fines not only affect the workability but also have

an effect on concretes hardened properties as well. During sampling it was found that the fines could easily be removed by continually shoveling the aggregate pile which allowed the fines settle to the bottom. The small amount of RCA at the bottom with the fines could be discarded. However, if this approach is followed in the future the elimination of the fines must also be considered in the gradation analysis. The elimination of fines may also help with achieving an acceptable ASTM C 67 aggregate, which was not achieved with this research due to the fines content.

If crushed waste glass is used in a concrete mixture it should be washed prior to use. This was not performed for this research and the results may have improved significantly if this was done. The presence of sugars and other contaminant from the beverages and food sources that these containers once held can have detrimental effects on a concrete. The literature review showed that washing these aggregates prior to use has a dramatic impact on the hardened properties of a concrete. However, the increased cost of washing must also be considered when analyzing the use of this material as an aggregate. The waste glass cullet collected from the recycling plant did not meet the requirements of ASTM C 33. This could be overcome by grading the waste glass prior to use and should be considered. The added cost of grading the glass to meet ASTM criteria would need to be considered if this approach is taken.

Future studies should investigate varying amounts of recycled aggregate on concretes with only PC. This will help determine at what percentage of replacement the recycled aggregates become detrimental. This research only investigated a 100% recycled aggregate with PC as the cementitious and the results showed a significant correlation between recycled aggregates and concrete quality. Although the detrimental effects were probably related primarily to ASR and waste glass, the effects on slump and other properties could be established and an optimum replacement amount could be found. Due to the strong influence PC has on the concrete industry, the use of recycled aggregates may be more acceptable and the

point at which these aggregates become detrimental, without the use of GGBFS, should be explored. The inclusion of powdered waste glass may also be beneficial. The literature review showed that the use of powdered glass significantly helped the ASR effects caused by the waste glass.

Future studies should investigate concretes composed of 100% GGBFS and varying replacement amounts of recycled aggregate. A concrete made with 100% GGBFS and 100% natural aggregate may have proved to be very successful. An optimum replacement of recycled aggregates could be established with this research. The GGBFS used should be a Grade 120, as this was shown to be a highly reactive slag and produced strength alone without the use of PC. Investigations could also be performed on different Grades of GGBFS as well. However, at replacement levels of 100% cementitious with GGBFS, the use of any Grade lower than 120 would probably result in decreased properties due to the lesser reactivity.

BIBLIOGRAPHY

- ACI. (2000). *ACI Committee Report 233R-95 (reapproved 2000), Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete*. Detroit: American Concrete Institute.
- ACI. (2001). *ACI Committee Report 555, Removal and Reuse of Hardened Concrete*. Detroit: American Concrete Institute.
- Ahmad Shayan, A. X. (2006). Performance of Glass Powder as a Pozolanic Material in Concrete: A Field Trial on Concrete Slabs. *Cement and Concrete Research*, 36, 457-468.
- ASDSO. (2011). *Fundamental Reinforced Concrete Design for Hydraulic Structures*. Baltimore: Association of State Dam Safety Officials.
- ASTM C 1202. (2010). *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*. West Conshohocken: American Society for Testing Materials.
- ASTM C 1260. (2007). *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*. West Conshohocken: American Society for Testing Materials.
- ASTM C 127. (2006). *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*. West Conshohocken: American Society for Testing Materials.
- ASTM C 128. (2006). *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate*. West Conshohocken: American Society for Testing Materials.
- ASTM C 136. (2006). *Standard Analysis of Fine Aggregate and Coarse Aggregates*. West Conshohocken: American Society for Testing Materials.
- ASTM C 1567. (2009). *Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate*. West Conshohocken: American Society for Testing Materials.

- ASTM C 33. (2008). *Standard Specification for Concrete Aggregates*. West Conshohocken: American Society for Testing Materials.
- ASTM C 666. (2008). *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*. West Conshohocken: American Society for Testing Materials.
- ASTM C 702. (2007). *Standard Practice for Reducing Samples of Aggregate to Testing Size*. West Conshohocken: American Society for Testing Materials.
- ASTM C-989. *Standard Specification for Slag Cement for Use in Concrete and Mortars*. West Conshohocken: American Society for Testing Materials.
- Bureau of Reclamation. (1988). *Concrete Manual, A Water Resources Technical Publication* (8 ed.). Washington DC: U.S. Department of the Interior, Bureau of Reclamation.
- Bureau of Reclamation. (2010). *Mass Concrete Mixture Proportioning Study for Folsom Dam Auxiliary Spillway Control Structure*. Denver: U.S. Department of Interior, Bureau of Reclamation.
- C. Meyer, N. E. (2001). Concrete with Waste Glass as Aggregate. *International Symposium Concrete Technology Unit of ASCE and University of Dundee* (pp. 1-9). Columbia University.
- D.S. Lane, C. O. (1999). *Combinations of Pozzolans and Ground Granulated Blast Furnace Slag for Durable Hydraulic Cement Concrete*. Charlottesville: Virginia Transportation Research Council & US Department of Transportation Federal Highway Administration.
- Detwiler, J. R. (2003). *PCA's Guide Specification for Concrete Subject to Alkali-Silica Reactions: Mitigation Measures*. Skokie: Portland Cement Association.
- en.wikipedia.org*. (2011). Retrieved April 3, 2011, from Wikipedia, the free Encyclopedia: http://en.wikipedia.org/wiki/Soda-lime_glass
- en.wikipedia.org*. (2011). Retrieved April 11, 2011, from Wikipedia, the free encyclopedia: http://en.wikipedia.org/wiki/Glass_recycling#cite_note-0
- Frearson, J. H. (1992). Sulfate Resistance of Mortars Containing Ground Granulated Blast Furnace Slag with Variable Alumina Content. *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete , II*, 1525-1542.

- Gholamreza Fathifazl, A. R. (2009, July 26). A Novel Method For Proportioning Structural Concrete Mixes Made with Recycled Concrete Aggregate. *Concrete International Magazine of ACI*.
- Hale, M. (2001). *The Effect of Ground Granulated Blast Furnace Slag on the Fresh and Hardened Properties of Concrete*. Norman: University of Oklahoma.
- Hogan, F. M. (1981). Evaluation for Durability and Strength Development of a Ground Granulated Blast Furnace Slag. *Cement, Concrete and Aggregates*, 3 (1), 40-52.
- Hooten, D. R. (2000). Canadian use of ground granulated blast-furnace slag as a supplementary cement material for the enhanced performance of concrete. *Canadian Journal of Civil Engineering* (27).
- Karthik Obla, H. K. (2007). *Crushed Returned Concrete as Aggregates for New Concrete*. RMC Research and Education Foundation, NRMCA Research Laboratory.
- M. Mageswari, B. V. (2010). The Use of Sheet Glass Powder as Fine Aggregate Replacement in Concrete. *The Open Civil Engineering Journal*, 65-71.
- Mayoor. (2009). *Recycled Reuse of Waste Materials in Civil Structures*. Pune, India: College of Engineering Pune.
- McLellen, D. H. (2009). *Effectiveness of Slag Cement in Preventing Alkali-Silica Reaction: Ten-Year Results*. Farmington Hills: American Concrete Institute, ACI SP-263-3.
- Mindess, Y. D. (2003). *Concrete* (2nd ed.). (M. Horton, Ed.) Upper Saddle River, New Jersey, United States: Pearson Education, Inc.
- Mirjana Malešev, V. R. (2010). Recycled Concrete as Aggregate for Structural Concrete Production. *Sustainability*.
- Osborne, G. (1989). *Carbonation and Permeability of Blast Furnace Slag Cement on Concretes from Field Structures*. Detroit: American Concrete Institute.
- PCA. (2005). *Design and Control of Concrete Mixtures* (14 ed.). Skokie: Portland Cement Association.
- Polley, C. (1996). *The Effects of Waste Glass Aggregate on the Strength and Durability of Portland Cement Concrete*. Madison: Department of Civil and Environmental Engineering, University of Wisconsin Madison.

- Reiner, M. (2007). *Technology, Environment, Resource and Policy Assessment of Sustainable Concrete in Urban Infrastructure*. Denver: University of Colorado Denver.
- Richardson, D. N. (2006). *Strength and Durability Characteristics of a 70% Ground Granulated Blast Furnace Slag Concrete Mix*. Rolla: Missouri Transportation Institute & Missouri Department of Transportation.
- Sakai, W. S. (1992). Properties of Granulated Blast Furnace Slag Cement Concrete. *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, II, 1367-1400.
- Shi, C. S. (1998, July-August). Effect of Supplementary Cementing Materials on the Specific Conductivity of Pore Solution and its Implications on the Rapid Chloride Permeability Test (AASHTO T277 and ASTM C1202). *ACI Materials Journal*, 389-394.
- Sippel, S. C. (2005). *Effects of Ground Granulated Blast Furnace Slag in Portland Cement Concrete*. University of Wisconsin-Madison, Department of Civil and Environmental Engineering. Madison: University of Wisconsin-Madison.
- Sivasundaram, V. a. (1992). Properties of Concrete Incorporating Low Quantity of Cement and High Volumes of Ground Granulated Slag. *ACI Materials Journal*, 89 (6).
- Stark, D. (1989). *Durability of Concrete in Sulfate Rich Soils*. Skokie: Portland Cement Association.
- U.S. Department of Transportation, Federal Highway Administration. (2004). Transportation Applications of Recycled Concrete Aggregate. *FHWA State of Practice National Review*.
- Verdugo, C. D. (2009). *The Practicality, Versatility and Feasibility of Utilizing Recycled Glass as a Concrete Aggregate*. Gainesville: University of Florida, Civil Engineering Department.
- Wang Ling, T. P. (2005). *Application of Ground Granulated Blast Furnace Slag in High Performance Concrete in China*. International Workshop on Sustainable Development and Concrete Technology.
- Weihua Jin, C. M. (2000). "Glascrete" Concrete with Glass Aggregate. *ACI Materials Journal*, 208-213.

World Buisness Council for Sustainable Development. (2002). *The Cement Sustainability Initiative, Our Agenda for Action*. Geneva, Switzerland: World Buisness Council for Sustainable Development.

www.glassproperties.com. (2011). Retrieved April 3, 2011, from *www.glassproperties.com*:
<http://glassproperties.com/glasses/>

www.ndconcrete.com. (n.d.). Retrieved February 25, 2011, from ND Ready Mix and Concrete Products Association, ND Chapter-Inc. ACPA:
<http://www.ndconcrete.com/article/using.html>

APPENDIX A

CC (control) Concrete Mixture Design (100% Cement & 100% Normal Aggregate) Batched on 3/27/2010					
Mix Proportion (SSD)					
Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	615.00	3.13	0.116	120	36.90
Fly Ash	0	0.00	0.000	100	0.00
BFS	0	0.00	0.000	85	0.00
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
UCD Rock	1646.00	10.11	0.374	18	14.81
UCD Sand	1323.94	8.07	0.299	18	11.92
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00		
Total Cost =					63.70
Mix Characteristics					
w/c	0.40				
Unit Weight (pcf)	141.9				
Cementitious material (lb)	615				
Suppl. Cementitious Mat.					
Fly Ash replacement (%)	0		0		
BFS replacement (%)	0		0		
Micron 3 (%)	0		0		
Silica Fume (%)	0.00		0.00		
Moisture Content					
UCD Sand pan	158.2	sand + pan wt.	889.7		
UCD Rock pan	113.4	rock + pan wt.	1349.9		
		dry wt. sand	860.5		
		dry wt. rock	1348.0		
UCD Sand mc (%)	4.16	mc-ssd	0.034577673		
UCD Rock mc (%)	0.15	mc-ssd	-0.00646104		
Batch Weights (yd³)					
Cement	615	lb			
Fly Ash	0	lb			
BFS	0	lb			
Micron 3	0	lb			
Silica Fume	0	lb			
UCD Rock	1635	lb			
UCD Sand	1370	lb			
Water	211	lb			
AEA	1.5	fl oz./cwt			
HRWR	5	fl oz./cwt			
Batch Weights (ft³)					
Batch size	1.95	cf			
Cement	44.3	lb			
Fly Ash	0.0	lb			
BFS	0.0	lb			
Micron 3	0.0	lb			
Silica Fume	0.0	lb			
UCD Rock	117.8	lb			
Material Properties					
Material	S.G.	A.C.			
Cement	3.15	-			
Fly Ash	2.67	-			
Micron 3	2.53	-			
BFS	2.93	-			
Silica Fume	2.20	-			
Rock	2.61	0.80			
Sand	2.63	0.70			
Testing Specimens Required					
Compressive cylinders	15	0.87			
RCIP cylinders	6	0.35			
MOR	0	0.00			
Unit weight	1	0.25			
Permeameter slabs	0	0.00			
Salt Pounding	0	0.00			
MOE	0	0.00			
F/T Beams	2	0.22			
Split Cylinder	0	0.00			
Total	1.69				
x 1.15	1.95				
Fresh Properties					
Slump	1.75	inch			
Empty Pot Vol.	0.25	ft ³			
Empty Pot Wt.	7.65	lb			
Full Pot Wt.	41.35	lb			
Unit Weight	134.80	lb/ft ³			

100-RA-C**Concrete Mixture Design (100% Cement & 100% Recycled Aggregate)**

Batched on 11/13/2010

Mix Proportion (SSD)

Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	615.00	3.13	0.116	120	36.90
Fly Ash	0	0.00	0.000	100	0.00
BFS	0	0.00	0.000	85	0.00
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
Recycled Concrete	1646.00	10.77	0.399	18	14.81
Waste Glass	1139.36	7.41	0.274	18	10.25
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
				Total Cost =	62.04

Material Properties

Material	S.G.	A.C.
Cement	3.15	-
Fly Ash	2.67	-
Micron 3	2.53	-
BFS	2.93	-
Silica Fume	2.20	-
Recycled Conc.	2.45	4.58
Waste Glass	2.47	0.181

Mix Characteristics

w/c	0.40
Unit Weight (pcf)	135.1
Cementitious material (lb)	615

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	0	0
Micron 3 (%)	0	0
Silica Fume (%)	0.00	0.00

Moisture Content

Waste Glass pan	100.7	F.A. + pan wt	270.4
Recycled Conc. pan	111.7	C.A. + pan wt	1114.8
		dry wt. F.A.	262.7
		dry wt. C.A.	1073.6
Waste Glass mc (%)	4.75	mc-ssd	0.045720864
Recycled Conc. mc (%)	4.28	mc-ssd	-0.002968105

Batch Weights (yd³)

Cement	615	lb
Fly Ash	0	lb
BFS	0	lb
Micron 3	0	lb
Silica Fume	0	lb
Recycled Concrete	1641	lb
Waste Glass	1191	lb
Water	199	lb
AEA	1.5	fl oz./cwt
HRWR	273	ml

Batch Weights (ft³)

Batch size	1.95	cf
Cement	44.3	lb
Fly Ash	0.0	lb
BFS	0.0	lb
Micron 3	0.0	lb
Silica Fume	0.0	lb
Recycled Concrete	118.3	lb
Waste Glass	85.8	lb
Water	14.3	lb
AEA	20.0	ml
HRWR	216.0	ml

Testing Specimens Required

Compressive cylinders	15	0.87
RCIP cylinders	6	0.35
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
F/T Beams	2	0.22
Split Cylinder	0	0.00
Total	1.69	
x 1.15	1.95	

Fresh Properties

Slump	0.00	inch
Empty Pot Vol.	0.249	ft ³
Empty Pot Wt.	7.85	lb
Full Pot Wt.	41.1	lb
Unit Weight	133.53	lb/ft ³
Air Content	4.0	%

RAC #2**Concrete Mixture Design (100% Cement & 100% Recycled Aggregate)**

Batched on 12/4/2010

Mix Proportion (SSD)

Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	615.00	3.13	0.116	120	36.90
Fly Ash	0	0.00	0.000	100	0.00
BFS	0	0.00	0.000	85	0.00
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
Recycled Concrete	1646.00	10.77	0.399	18	14.81
Waste Glass	1139.36	7.41	0.274	18	10.25
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
Total Cost =					62.04

Material Properties

Material	S.G.	A.C.
Cement	3.15	-
Fly Ash	2.67	-
Micron 3	2.53	-
BFS	2.93	-
Silica Fume	2.20	-
Recycled Conc.	2.45	4.58
Waste Glass	2.47	0.181

Mix Characteristics

w/c	0.40
Unit Weight (pcf)	135.1
Cementitious material (lb)	615

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	0	0
Micron 3 (%)	0	0
Silica Fume (%)	0.00	0.00

Moisture Content

Waste Glass pan	111.7	F.A. + pan wt.	850.4
Recycled Conc. pan	113.5	C.A. + pan wt.	843.8
		dry wt. F.A.	819.1
		dry wt. C.A.	823.8
Waste Glass mc (%)	4.42	mc-ssd	0.042436537
Recycled Conc. mc (%)	2.82	mc-ssd	-0.017642883

Batch Weights (yd³)

Cement	615	lb
Fly Ash	0	lb
BFS	0	lb
Micron 3	0	lb
Silica Fume	0	lb
Recycled Concrete	1617	lb
Waste Glass	1188	lb
Water	227	lb
AEA	1.5	fl oz./cwt
HRWR	23	fl oz./cwt

Batch Weights (ft³)

Batch size	1.95	cf
Cement	44.3	lb
Fly Ash	0.0	lb
BFS	0.0	lb
Micron 3	0.0	lb
Silica Fume	0.0	lb
Recycled Concrete	116.5	lb
Waste Glass	85.6	lb
Water	16.3	lb
AEA	20.0	ml
HRWR	316.0	ml

Testing Specimens Required

Compressive cylinders	15	0.87
RCIP cylinders	6	0.35
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
F/T Beams	2	0.22
Split Cylinder	0	0.00
Total		1.69
x 1.15		1.95

Fresh Properties

Slump	0.50	inch
Empty Pot Vol.	0.249	ft ³
Empty Pot Wt.	7.85	lb
Full Pot Wt.	42.5	lb
Unit Weight	139.16	lb/ft ³
Air Content	5.2	%

100-RA-BF**Concrete Mixture Design (100% GGBFS & 100% Recycled Aggregate)**

Batched on 11/13/2010

Mix Proportion (SSD)

Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	0.00	0.00	0.000	120	0.00
Fly Ash	0	0.00	0.000	100	0.00
BFS	615	3.36	0.125	85	26.14
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
Recycled Concrete	1646.00	10.77	0.399	18	14.81
Waste Glass	1103.22	7.17	0.266	18	9.93
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
Total Cost =					50.95

Material Properties

Material	S.G.	A.C.
Cement	3.15	-
Fly Ash	2.67	-
Micron 3	2.53	-
BFS	2.93	-
Silica Fume	2.20	-
Recycled Conc.	2.45	4.58
Waste Glass	2.47	0.181

Mix Characteristics

w/c	0.40
Unit Weight (pcf)	133.7
Cementitious material (lb)	615

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	100	615
Micron 3 (%)	0	0
Silica Fume (%)	0.00	0.00

Moisture Content

Waste Glass pan	100.7	F.A. + pan wt	270.4
Recycled Conc. pan	111.7	C.A. + pan wt	1114.8
		dry wt. F.A.	262.7
		dry wt. C.A.	1073.6

Waste Glass mc (%)	4.75	mc-ssd	0.045720864
Recycled Conc. mc (%)	4.28	mc-ssd	-0.002968105

Batch Weights (yd³)

Cement	0	lb
Fly Ash	0	lb
BFS	615	lb
Micron 3	0	lb
Silica Fume	0	lb
Recycled Concrete	1641	lb
Waste Glass	1154	lb
Water	200	lb
AEA	1.5	fl oz/cwt
HRWR	56	fl oz/cwt

Batch Weights (ft³)

Batch size	1.95	cf
Cement	0.0	lb
Fly Ash	0.0	lb
BFS	44.3	lb
Micron 3	0.0	lb
Silica Fume	0.0	lb
Recycled Concrete	118.3	lb
Waste Glass	83.1	lb
Water	14.4	lb
AEA	20.0	ml
HRWR	740.0	ml

Testing Specimens Required

Compressive cylinders	15	0.87
RCIP cylinders	6	0.35
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Pounding	0	0.00
MOE	0	0.00
F/T Beams	2	0.22
Split Cylinder	0	0.00
Total		1.69
x 1.15		1.95

Fresh Properties

Slump	0.75	inch
Empty Pot Vol.	0.249	ft ³
Empty Pot Wt.	7.85	lb
Full Pot Wt.	42.9	lb
Unit Weight	140.76	lb/ft ³
Air Content	4.6	%

75-RA-BF**Concrete Mixture Design (75% BFS , 75% Recycled Aggregate)**

Batched on 11/20/2010

Mix Proportion (SSD)

Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	153.75	0.78	0.029	120	9.23
Fly Ash	0	0.00	0.000	100	0.00
BFS	461.25	2.52	0.093	85	19.60
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
UCD Rock	400.00	2.46	0.091	18	3.60
Recycled Concrete	1200.00	7.85	0.291	10	6.00
UCD Sand	315.60	1.92	0.071	18	2.84
Waste Glass	887.40	5.77	0.214	8	3.55
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
				Total Cost	44.89

Mix Characteristics

w/c	0.40
Unit Weight (pcf)	135.7
Cementitious material (lb)	615

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	75	461.25
Micron 3 (%)	0	0
Silica Fume (%)	0.00	0.00

Moisture Content, Normal Aggregates

UCD Sand pan	111.7	sand + pan wt.	956.4
UCD Rock pan	113.6	rock + pan wt.	1481.9
		dry wt. sand	943.3
		dry wt. rock	1480.1

Moisture Content, Recycled Aggregates

Waste Glass pan	103.0	F.A. + pan wt.	824.3
Recycled Conc. pan	121.6	C.A. + pan wt.	1178.2
		dry wt. F.A.	794.3
		dry wt. C.A.	1148.7

UCD sand mc (%)	1.58	mc-ssd	0.0088
UCD rock mc (%)	0.13	mc-ssd	-0.0067
Waste Glass mc (%)	4.34	mc-ssd	0.0416
Recycled Conc. mc (%)	2.87	mc-ssd	-0.0171

Batch Weights (yd³)

Cement	154	lb
Fly Ash	0	lb
BFS	461	lb
Micron 3	0	lb
Silica Fume	0	lb
UCD Rock	397	lb
Recycled Concrete	1180	lb
UCD Sand	318	lb
Waste Glass	924	lb
Water	230	lb
AE/A	1.5	fl oz./cwt
HRWR	27	fl oz./cwt

Batch Weights (ft³)

Batch size	1.95	cf
Cement	11.1	lb
Fly Ash	0.0	lb
BFS	33.2	lb
Micron 3	0.0	lb
Silica Fume	0.0	lb
UCD Rock	28.6	lb
Recycled Concrete	85.0	lb
UCD Sand	22.9	lb
Waste Glass	66.6	lb
Water	16.5	lb
AE/A	20.0	ml
HRWR	350.0	ml

Material Properties

Material	S.G.	A.C.
Cement	3.15	-
Fly Ash	2.67	-
Micron 3	2.53	-
BFS	2.93	-
Silica Fume	2.20	-
Rock	2.61	0.80
Recycled Conc.	2.45	4.58
Sand	2.63	0.70
Waste Glass	2.47	0.18

Testing Specimens Required

Compressive cylinders	15	0.87
RCIP cylinders	6	0.35
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
F/T Beams	2	0.22
Split Cylinder	0	0.00
Total	1.69	
x 1.15	1.95	

Fresh Properties

Slump	3.00	inch
Empty Pot Vol.	0.249	ft ³
Empty Pot Wt.	7.9	lb
Full Pot Wt.	41.3	lb
Unit Weight	134.14	lb/ft ³
Air Content	7.5	%

50-RA-BF**Concrete Mixture Design (50% BFS , 50% Recycled Aggregate)**

Batched on 3/27/2010

Mix Proportion (SSD)

Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	307.50	1.56	0.058	120	18.45
Fly Ash	0	0.00	0.000	100	0.00
BFS	307.5	1.68	0.062	85	13.07
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
UCD Rock	800.00	4.91	0.182	18	7.20
Recycled Concrete	800.00	5.23	0.194	10	4.00
UCD Sand	649.18	3.96	0.147	18	5.84
Waste Glass	608.46	3.96	0.147	8	2.43
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
Total Cost =					51.07

Mix Characteristics

w/c	0.40
Unit Weight (pcf)	137.7
Cementitious material (lb)	615

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	50	307.5
Micron 3 (%)	0	0
Silica Fume (%)	0.00	0.00

Moisture Content, Normal Aggregates

UCD Sand pan	158.2	sand + pan wt.	889.7
UCD Rock pan	113.4	rock + pan wt.	1349.9
		dry wt. sand	860.5
		dry wt. rock	1348.0

Moisture Content, Recycled Aggregates

Waste Glass pan	111.9	sand + pan wt.	727.3
Recycled Conc. pan	121.5	rock + pan wt.	1325.8
		dry wt. sand	707.9
		dry wt. rock	1291.2

UCD Sand mc (%)	4.16	mc-ssd	0.0346
UCD Rock mc (%)	0.15	mc-ssd	-0.0065
Waste Glass mc (%)	3.26	mc-ssd	0.0307
Recycled Conc. mc (%)	2.96	mc-ssd	-0.0162

Batch Weights (yd³)

Cement	308	lb
Fly Ash	0	lb
BFS	308	lb
Micron 3	0	lb
Silica Fume	0	lb
UCD Rock	795	lb
Recycled Concrete	787	lb
UCD Sand	672	lb
Waste Glass	627	lb
Water	223	lb
ATA	1.5	fl oz./cwt
HRWR	5	fl oz./cwt

Batch Weights (ft³)

Batch size	1.95	cf
Cement	22.2	lb
Fly Ash	0.0	lb
BFS	22.2	lb
Micron 3	0.0	lb
Silica Fume	0.0	lb
UCD Rock	57.3	lb
Recycled Concrete	56.7	lb
UCD Sand	48.4	lb
Waste Glass	45.2	lb
Water	16.1	lb
ATA	20.0	ml
HRWR	60.0	ml

Material Properties

Material	S.G.	A.C.
Cement	3.15	-
Fly Ash	2.67	-
Micron 3	2.53	-
BFS	2.93	-
Silica Fume	2.20	-
Rock	2.61	0.80
Recycled Conc.	2.45	4.58
Sand	2.63	0.70
Waste Glass	2.47	0.18

Testing Specimens Required

Compressive cylinders	15	0.87
RCIP cylinders	6	0.35
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
F/T Beams	2	0.22
Split Cylinder	0	0.00
Total		1.69
x 1.15		1.95

Fresh Properties

Slump	0.50	inch
Empty Pot Vol.	0.25	ft ³
Empty Pot Wt.	7.6	lb
Full Pot Wt.	42.8	lb
Unit Weight	140.80	lb/ft ³
Air Content	7.0	%

25-RA-BF					
Concrete Mixture Design (25% BFS , 25% Recycled Aggregate)					
Batched on 11/20/2010					
Mix Proportion (SSD)					
Material	Weight	Volume (cf)	Volume Check	Unit Cost (\$/ton)	Amount (\$)
Cement	461.25	2.35	0.087	120	27.68
Fly Ash	0	0.00	0.000	100	0.00
BFS	153.75	0.84	0.031	85	6.53
Micron 3	0	0.00	0.000	500	0.00
Silica Fume	0	0.00	0.000	500	0.00
UCD Rock	1200.00	7.37	0.273	18	10.80
Recycled Concrete	400.00	2.62	0.097	10	2.00
UCD Sand	1000.75	6.10	0.226	18	9.01
Waste Glass	312.65	2.03	0.075	8	1.25
Water	246.00	3.94	0.146	0.6	0.07
Air	0.065	1.76	0.065	-	-
		27.00	1.00	-	-
				Total Cost	57.34
Mix Characteristics					
w/c		0.40			
Unit Weight (pcf)		139.8			
Cementitious material (lb)		615			
Suppl. Cementitious Mat.		Percent (%)	Weight (lb)		
Fly Ash replacement (%)		0	0		
BFS replacement (%)		25	153.75		
Micron 3 (%)		0	0		
Silica Fume (%)		0.00	0.00		
Moisture Content, Normal Aggregates					
UCD Sand pan	111.7	sand + pan wt.	956.4		
UCD Rock pan	113.6	rock + pan wt.	1481.9		
		dry wt. sand	943.3		
		dry wt. rock	1480.1		
Moisture Content, Recycled Aggregates					
Waste Glass pan	103.0	F.A. + pan wt.	824.3		
Recycled Conc. pan	121.6	C.A. + pan wt.	1178.2		
		dry wt. F.A.	794.3		
		dry wt. C.A.	1148.7		
UCD Sand mc (%)	1.58	mc-ssd	0.0088		
UCD Rock mc (%)	0.13	mc-ssd	-0.0067		
Waste Glass mc (%)	4.34	mc-ssd	0.0416		
Recycled Conc. mc (%)	2.87	mc-ssd	-0.0171		
Batch Weights (yd³)					
Cement	461	lb			
Fly Ash	0	lb			
BFS	154	lb			
Micron 3	0	lb			
Silica Fume	0	lb			
UCD Rock	1192	lb			
Recycled Concrete	393	lb			
UCD Sand	1010	lb			
Waste Glass	326	lb			
Water	239	lb			
AE/A	1.5	fl oz / cwt			
HRWR	27	fl oz / cwt			
Batch Weights (ft³)					
Batch size	1.95	cf			
Cement	33.2	lb			
Fly Ash	0.0	lb			
BFS	11.1	lb			
Micron 3	0.0	lb			
Silica Fume	0.0	lb			
UCD Rock	85.9	lb			
Recycled Concrete	28.3	lb			
UCD Sand	72.7	lb			
Waste Glass	23.5	lb			
Water	17.2	lb			
AE/A	20.0	ml			
HRWR	350.0	ml			
Material Properties					
Material	S.G.	A.C.			
Cement	3.15	-			
Fly Ash	2.67	-			
Micron 3	2.53	-			
BFS	2.93	-			
Silica Fume	2.20	-			
Rock	2.61	0.80			
Recycled Conc.	2.45	4.58			
Sand	2.63	0.70			
Waste Glass	2.47	0.18			
Testing Specimens Required					
Compressive cylinders	15	0.87			
RCIP cylinders	6	0.35			
MOR	0	0.00			
Unit weight	1	0.25			
Permeameter slabs	0	0.00			
Salt Ponding	0	0.00			
MOE	0	0.00			
F/T Beams	2	0.22			
Split Cylinder	0	0.00			
	Total	1.69			
	x 1.15	1.95			
Fresh Properties					
Slump	8.25	inch			
Empty Pot Vol.	0.249	ft ³			
Empty Pot Wt.	7.85	lb			
Full Pot Wt.	42.35	lb			
Unit Weight	138.55	lb/ft ³			
Air Content	6.4	%			

APPENDIX B

 Sevenson Inc. / Paving Systems 1925 N. Main Street Denver, CO 80234 303.975.9955 Fax 303.675.9556		LABORATORY TEST REPORT CLIENT: Boskewy Concrete SOURCE: Brighton SAMPLED BY: Client PROJECT: Miscellaneous		Was Test Project No. 032408 REPORT DATE: March 5, 2003	
MATERIAL ASTM C 30.50 Fine Aggregate		ASTM C 30.50 Fine Aggregate January 28, 2003 Site: 100			
DESCRIPTION DATE SAMPLE LOCATION		ASTM C 30.50 Fine Aggregate January 28, 2003 Site: 100			
Approved: Tested in accordance with Quality Test Plan (QTP) of M. A. AASHTO 1997 Specification					
ASTM C 117 & 136, AASHTO T 27 1.5% to 3.0% 1.5% to 3.0% 1.5% to 3.0%		ASTM C 128, AASHTO T 84, Brick Sound Crack = 2.01, Bulk Specific Gravity = 1.67, Dry Apparent Specific Gravity = 1.67, Absorption = 0.7%		ASTM C 140, AASHTO T 140, Medium Gravel 5.0% to 10.0%	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 4.0 4.0	
4.0 4.0 4.0		4.0 4.0 4.0		4.0 	

WEST

CLIENT: Bestway Concrete
SOURCE: Brighton
SAMPLED BY: Client
PROJECT: Miscellaneous

WASTES PROJECT NO.: 202409
REPORT DATE: March 2025

PROJECT: Miscellaneous

303.975.9059, Fax 303.975.9069

[illegible]

1. *Introduction*
 2. *Background*
 3. *Methods*
 4. *Results*
 5. *Discussion*
 6. *Conclusion*
 7. *References*
 8. *Appendix*
 9. *Tables*
 10. *Figures*
 11. *Supplementary Materials*
 12. *Correspondence*
 13. *Conflict of Interest*
 14. *Acknowledgments*
 15. *Author Contributions*
 16. *References*
 17. *Appendix*
 18. *Tables*
 19. *Figures*
 20. *Supplementary Materials*
 21. *Correspondence*
 22. *Conflict of Interest*
 23. *Acknowledgments*
 24. *Author Contributions*
 25. *References*
 26. *Appendix*
 27. *Tables*
 28. *Figures*
 29. *Supplementary Materials*
 30. *Correspondence*
 31. *Conflict of Interest*
 32. *Acknowledgments*
 33. *Author Contributions*
 34. *References*
 35. *Appendix*
 36. *Tables*
 37. *Figures*
 38. *Supplementary Materials*
 39. *Correspondence*
 40. *Conflict of Interest*
 41. *Acknowledgments*
 42. *Author Contributions*
 43. *References*
 44. *Appendix*
 45. *Tables*
 46. *Figures*
 47. *Supplementary Materials*
 48. *Correspondence*
 49. *Conflict of Interest*
 50. *Acknowledgments*
 51. *Author Contributions*
 52. *References*
 53. *Appendix*
 54. *Tables*
 55. *Figures*
 56. *Supplementary Materials*
 57. *Correspondence*
 58. *Conflict of Interest*
 59. *Acknowledgments*
 60. *Author Contributions*
 61. *References*
 62. *Appendix*
 63. *Tables*
 64. *Figures*
 65. *Supplementary Materials*
 66. *Correspondence*
 67. *Conflict of Interest*
 68. *Acknowledgments*
 69. *Author Contributions*
 70. *References*
 71. *Appendix*
 72. *Tables*
 73. *Figures*
 74. *Supplementary Materials*
 75. *Correspondence*
 76. *Conflict of Interest*
 77. *Acknowledgments*
 78. *Author Contributions*
 79. *References*
 80. *Appendix*
 81. *Tables*
 82. *Figures*
 83. *Supplementary Materials*
 84. *Correspondence*
 85. *Conflict of Interest*
 86. *Acknowledgments*
 87. *Author Contributions*
 88. *References*
 89. *Appendix*
 90. *Tables*
 91. *Figures*
 92. *Supplementary Materials*
 93. *Correspondence*
 94. *Conflict of Interest*
 95. *Acknowledgments*
 96. *Author Contributions*
 97. *References*
 98. *Appendix*
 99. *Tables*
 100. *Figures*
 101. *Supplementary Materials*
 102. *Correspondence*
 103. *Conflict of Interest*
 104. *Acknowledgments*
 105. *Author Contributions*
 106. *References*
 107. *Appendix*
 108. *Tables*
 109. *Figures*
 110. *Supplementary Materials*
 111. *Correspondence*
 112. *Conflict of Interest*
 113. *Acknowledgments*
 114. *Author Contributions*
 115. *References*
 116. *Appendix*
 117. *Tables*
 118. *Figures*
 119. *Supplementary Materials*
 120. *Correspondence*
 121. *Conflict of Interest*
 122. *Acknowledgments*
 123. *Author Contributions*
 124. *References*
 125. *Appendix*
 126. *Tables*
 127. *Figures*
 128. *Supplementary Materials*
 129. *Correspondence*
 130. *Conflict of Interest*
 131. *Acknowledgments*
 132. *Author Contributions*
 133. *References*
 134. *Appendix*
 135. *Tables*
 136. *Figures*
 137. *Supplementary Materials*
 138. *Correspondence*
 139. *Conflict of Interest*
 140. *Acknowledgments*
 141. *Author Contributions*
 142. *References*
 143. *Appendix*
 144. *Tables*
 145. *Figures*
 146. *Supplementary Materials*
 147. *Correspondence*
 148. *Conflict of Interest*
 149. *Acknowledgments*
 150. *Author Contributions*
 151. *References*
 152. *Appendix*
 153. *Tables*
 154. *Figures*
 155. *Supplementary Materials*
 156. *Correspondence*
 157. *Conflict of Interest*
 158. *Acknowledgments*
 159. *Author Contributions*
 160. *References*
 161. *Appendix*
 162. *Tables*
 163. *Figures*
 164. *Supplementary Materials*
 165. *Correspondence*
 166. *Conflict of Interest*
 167. *Acknowledgments*
 168. *Author Contributions*
 169. *References*
 170. *Appendix*
 171. *Tables*
 172. *Figures*
 173. *Supplementary Materials*
 174. *Correspondence*
 175. *Conflict of Interest*
 176. *Acknowledgments*
 177. *Author Contributions*
 178. *References*
 179. *Appendix*
 180. *Tables*
 181. *Figures*
 182. *Supplementary Materials*
 183. *Correspondence*
 184. *Conflict of Interest*
 185. *Acknowledgments*
 186. *Author Contributions*
 187. *References*
 188. *Appendix*
 189. *Tables*
 190. *Figures*
 191. *Supplementary Materials*
 192. *Correspondence*
 193. *Conflict of Interest*
 194. *Acknowledgments*
 195. *Author Contributions*
 196. *References*
 197. *Appendix*
 198. *Tables*
 199. *Figures*
 200. *Supplementary Materials*
 201. *Correspondence*
 202. *Conflict of Interest*
 203. *Acknowledgments*
 204. *Author Contributions*
 205. *References*
 206. *Appendix*
 207. *Tables*
 208. *Figures*
 209. *Supplementary Materials*
 210. *Correspondence*
 211. *Conflict of Interest*
 212. *Acknowledgments*
 213. *Author Contributions*
 214. *References*
 215. *Appendix*
 216. *Tables*
 217. *Figures*
 218. *Supplementary Materials*
 219. *Correspondence*
 220. *Conflict of Interest*
 221. *Acknowledgments*
 222. *Author Contributions*
 223. *References*
 224. *Appendix*
 225. *Tables*
 226. *Figures*
 227. *Supplementary Materials*
 228. *Correspondence*
 229. *Conflict of Interest*
 230. *Acknowledgments*
 231. *Author Contributions*
 232. *References*
 233. *Appendix*
 234. *Tables*
 235. *Figures*
 236. *Supplementary Materials*
 237. *Correspondence*
 238. *Conflict of Interest*
 239. *Acknowledgments*
 240. *Author Contributions*
 241. *References*
 242. *Appendix*
 243. *Tables*
 244. *Figures*
 245.

[illegible]

COMMENTS

Recycled Concrete Coarse Aggregate (RCA) Gradation #1 (2/12/10)									
Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
1-1/2"	37.5	553.0	553.0	0.00	2700	0.00	0.00	0.00	100
1"	25	550.6	550.6	0.00	1800	0.00	0.00	0.00	100
3/4"	19	557.0	603.2	46.20	1400	46.20	2.30	2.30	98
1/2"	12.5	544.5	1336.6	792.10	890	838.30	39.48	41.78	58
3/8"	9.5	547.0	951.1	404.10	670	1242.4	20.14	61.92	38
No. 4	4.75	507.8	932.6	424.80	330	1667.20	21.17	83.09	17
No. 8	2.36	493.4	638.3	144.90	200	1812.10	7.22	90.31	10
No. 16	1.18	424.2	489.4	65.20	200	1877.30	3.25	93.56	6
Pan	0	367.1	495.0	127.90	-	2005.20	6.37	99.94	0
Original Weight Wo = 2006.5 grams Final Weight Wf = 2005.2 grams Percent Loss % Loss = 0.06 % Maximum Agg. Size Mas = 1.00 inch Nom. Max. Agg. Size NMAS = 3/4 inch									

Recycled Concrete Coarse Aggregate (RCA) Gradation #2 (2/12/10)									
Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
1-1/2"	37.5	553	553	0.00	2700	0.00	0.00	0.00	100
1"	25	550.6	550.6	0.00	1800	0.00	0.00	0.00	100
3/4"	19	557.1	572	14.90	1400	14.90	0.63	0.63	99
1/2"	12.5	544.6	1321.9	777.30	890	792.20	32.98	33.61	66
3/8"	9.5	547	1118.3	571.30	670	1363.50	24.24	57.85	42
No. 4	4.75	507.9	1104.2	596.30	330	1959.80	25.30	83.14	17
No. 8	2.36	493.4	699	205.60	200	2165.40	8.72	91.87	8
No. 16	1.18	424.2	511.2	87.00	200	2252.40	3.69	95.56	4
Pan	0	367.1	470.5	103.40	-	2355.80	4.39	99.94	0
Original Weight Wo = 2357.1 grams Final Weight Wf = 2355.8 grams Percent Loss % Loss = 0.06 % Maximum Agg. Size MAS = 1.00 inch Nom. Max. Agg. Size NMAS = 3/4 inch									

Recycled Concrete Coarse Aggregate (RCA) Gradation #3 (2/17/10)

Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
1-1/2"	37.5	553.0	553.0	0.00	2700	0.00	0.00	0.00	100
1"	25	550.6	550.6	0.00	1800	0.00	0.00	0.00	100
3/4"	19	557.2	581.3	24.10	1400	24.10	1.25	1.25	99
1/2"	12.5	544.6	1339.5	794.90	890	819.00	41.30	42.56	57
3/8"	9.5	547.1	962.0	414.90	670	1233.90	21.56	64.12	36
No. 4	4.75	508.0	947.4	439.40	330	1673.30	22.83	86.95	13
No. 8	2.36	493.3	614.8	121.50	200	1794.80	6.31	93.26	7
No. 16	1.18	424.1	479.6	55.50	200	1850.30	2.88	96.14	4
Pan	0	367.1	440.9	73.80	-	1924.10	3.83	99.98	0
Original Weight Wo = 1924.5 grams Final Weight Wf = 1924.1 grams Percent Loss % Loss = 0.02 % Maximum Agg. Size Mas = 1.00 inch Nom. Max. Agg. Size NMAS = 3/4 inch									

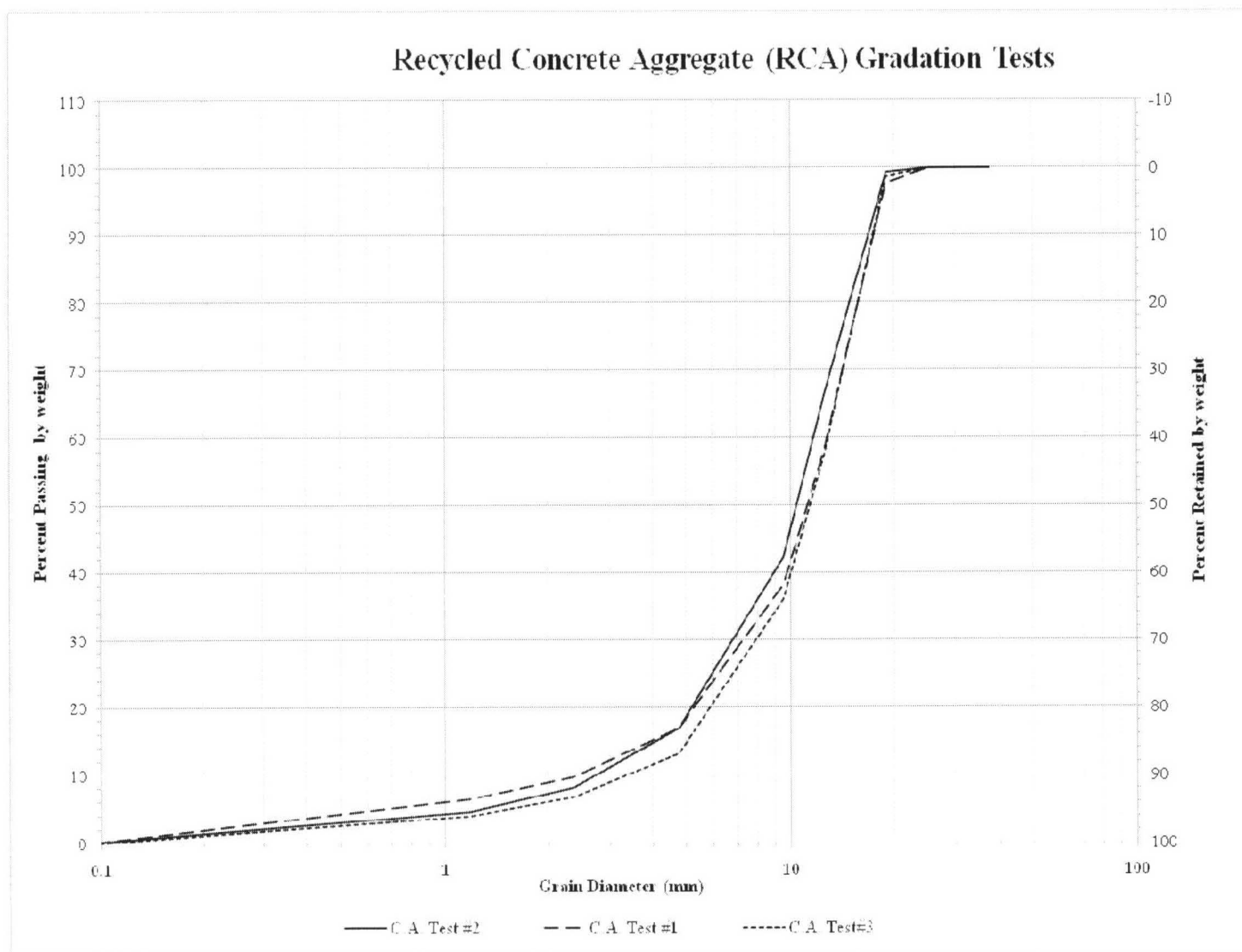
Recycled Concrete Unit Weight Test #1 (3/5/10)					
Compact Unit Weight:					
Wt. of Measure	M =	7.60	lb	3448.40	grams
Wt. of M and Compact Sample	Msc =	29.36	lb	13315.50	grams
Wt. of Wet Compact Sample	Scw =	21.75	lb	9867.10	grams
Wt. of Dry Compact Sample	Scd =	21.25	lb	9640.77	grams
Loose Unit Weight:					
Wt. of Measure	M =	7.60	lb	3448.40	grams
Wt. of M and Loose Sample	MSI =	27.26	lb	12365.50	grams
Wt. of Wet Loose Sample	Slw =	19.66	lb	8917.10	grams
Wt. of Dry Loose Sample	Sld =	19.21	lb	8712.56	grams
Moisture Content:					
Wt. of Beaker	B =	515.8	grams		
Wt. of Beaker and Wet Sample	Bsw =	742.5	grams		
Wt. of Beaker and Dry Sample	BSd =	737.3	grams		
Wt. of Wet Sample	Sw =	226.7	grams		
Wt. of Dry Sample	Sd =	221.5	grams		
Data Results:					
Moisture Content	MC =	2.35	%		
Dry Compact Unit Weight	DRUW =	85.02	lb/ft ³		
Dry Loose Unit Weight	DLUW =	76.83	lb/ft ³		

Recycled Concrete Aggregate (RCA) Specific Gravity Test #1 (2/19/10)		
Weight of Beaker, Be	602.40	grams
Weight of Beaker & Sample, Be (s)	996.50	grams
Weight of Sample, S1	394.10	grams
Weight of Beaker, Sample, & Water, Be (sw)	1309.60	grams
Weight of Beaker & Water, Be (w)	1077.70	grams
Results :		
Volume of Solids	2.6	ft ³
Density of Sample	151.6	lb/ft ³
Specific Gravity, SG	2.43	

Absorption Capacity of Recycled Concrete Aggregate (RCA) Test #1 (2/19/10)		
Weight of Bowl, Bo	171.40	grams
Weight of Bowl & SSD Sample, Bo (ssd)	1323.00	grams
Weight of Bowl & OD Sample, Bo (od)	1271.40	grams
Weight of SSD Sample, SSD wt	1151.60	grams
Weight of OD Sample, OD wt	1100.00	grams
Results :		
Absorption Capacity	4.69	%

Recycled Concrete Specific Gravity Test #2 (3/11/10)		
Weight of Beaker, Be	465.10	grams
Weight of Beaker & Sample, Be (s)	947.10	grams
Weight of Sample, S1	482.00	grams
Weight of Beaker, Sample, & Water, Be (sw)	1740.00	grams
Weight of Beaker & Water, Be (w)	1453.60	grams
Results :		
Volume of Solids	3.1	ft ³
Density of Sample	153.8	lb/ft ³
Specific Gravity, SG	2.46	

Absorption Capacity of Recycled Concrete Test #2 (3/11/10)		
Weight of Bowl, Bo	182.00	grams
Weight of Bowl & SSD Sample, Bo (ssd)	542.10	grams
Weight of Bowl & OD Sample, Bo (od)	526.70	grams
Weight of SSD Sample, SSD wt	360.10	grams
Weight of OD Sample, OD wt	344.70	grams
Results :		
Absorption Capacity	4.47	%



Waste Glass Aggregate Gradation #1 (2/19/10)

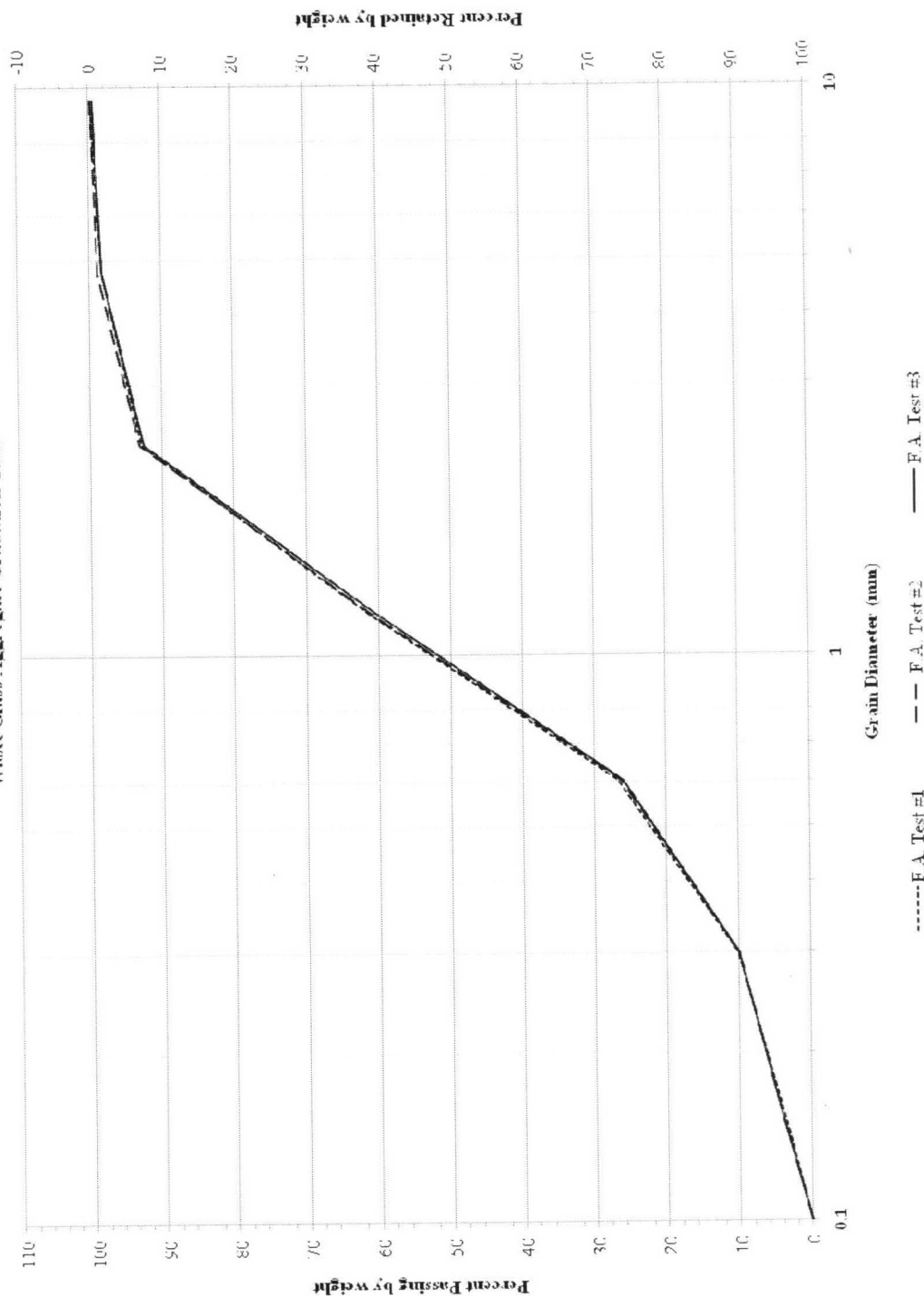
Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
3/8"	9.5	547.0	551.0	4.00	670	4.00	0.24	0.24	100
4	4.75	508.2	531.0	22.80	330	26.80	1.39	1.63	98
8	2.36	493.4	582.3	88.90	200	115.70	5.41	7.04	93
16	1.18	424.2	946.5	522.30	200	638.00	31.78	38.82	61
30	0.6	390.6	958.3	567.70	200	1205.70	34.55	73.37	27
50	0.3	370.8	642.8	272.00	200	1477.70	16.55	89.92	10
100	0.15	347.1	453.9	106.80	200	1584.50	6.50	96.42	4
Pan	0	367.1	425.1	58.00	-	1642.50	3.53	99.95	0
Original Weight Wo = 1643.30 grams									
Final Weight Wf = 1642.50 grams									
Percent Loss % Loss = 0.05 %									
Fineness Modulus FM = 4.07 in.									

Waste Glass Aggregate Gradation #2 (2/19/10)

Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
3/8"	9.5	547.1	549.7	2.60	670	2.60	0.16	0.16	100
4	4.75	508.1	523.0	14.90	330	17.50	0.94	1.11	99
8	2.36	493.5	583.9	90.40	200	107.90	5.72	6.82	93
16	1.18	424.2	934.6	510.40	200	618.30	32.27	39.09	61
30	0.6	390.5	938.7	548.20	200	1166.50	34.66	73.75	26
50	0.3	370.7	627.5	256.80	200	1423.30	16.23	89.98	10
100	0.15	346.8	444.7	97.90	200	1521.20	6.19	96.17	4
Pan	0	367.1	427.6	60.50	-	1581.70	3.82	99.99	0
Original Weight Wo = 1581.80 grams									
Final Weight Wf = 1581.70 grams									
Percent Loss % Loss = 0.01 %									
Fineness Modulus FM = 4.07 in.									

Waste Glass Aggregate Gradation #3 (2/19/10)									
Sieve Size	Sieve Size	Sieve Weight	Sieve & Sample Weight	Sample Weight	ASTM Maximum Allowable	Cumml. Sample Weight	Percent Retained	Cumml. Percent Retained	Percent Passing
(Std)	(mm)	(grams)	(grams)	(grams)	(grams)	(grams)	(%)	(%)	(%)
3/8"	9.5	547.0	556.0	9.00	670	9.00	0.54	0.54	99
4	4.75	507.9	527.0	19.10	330	28.10	1.15	1.70	98
8	2.36	493.3	585.8	92.50	200	120.60	5.59	7.29	93
16	1.18	424.2	962.5	538.30	200	658.9	32.52	39.81	60
30	0.6	390.6	957.9	567.30	200	1226.2	34.27	74.08	26
50	0.3	370.7	635.2	264.50	200	1490.7	15.98	90.06	10
100	0.15	346.9	448.1	101.20	200	1591.9	6.11	96.17	4
Pan	0	367.2	431.3	64.10	-	1656.0	3.87	100.04	0
Original Weight Wo = 1655.30 grams									
Final Weight Wf = 1656.00 grams									
Percent Loss % Loss = -0.04 %									
Fineness Modulus FM = 4.10 in.									

Waste Glass Aggregate Gradation Tests



Specific Gravity and Absorption Capacity of Recycled Glass Test #1 (2/25/10)			
Wt. of Pycnometer and Water	PW =	673.10	grams
Wt. of SSD Sample	S ssd =	277.30	grams
Wt. of Pycnometer, Water, and Sample	PWS =	837.90	grams
Wt. of Bowl	B =	171.10	grams
Wt. of Bowl and Dry Sample	BS dry =	447.90	grams
Wt. of Dry Sample	S dry =	276.80	grams
Results :			
Bulk Specific Gravity (Dry)	BSG dry =	2.460	
Bulk Specific Gravity (SSD)	BSG ssd =	2.465	
Absolute Specific Gravity	ASG =	2.471	
Absorption Capacity	AC =	0.181	%

LEHIGH

HEIDELBERGCEMENT Group

TECHNICAL SERVICES

SALES & MARKETING

2300 Clayton Rd., Suite 300
Concord, CA 94520
Telephone (925) 521-3601

PLANT LOCATION

Lehigh Hanson West

LIN 08-05-0483

(UHV ID: 0804735-New Location)

ALLCEM GGBFS (SLAG CEMENT) TEST REPORT

Specification: ASTM C 989-05 Grades 100 & 120

Report Date: 9-Sep-08

STANDARD CHEMICAL AND PHYSICAL REQUIREMENTS	TEST RESULTS	ASTM C 989 SPECIFICATIONS	
		Grade 100	Grade 120
Sulfur Trioxide (SO ₃), %	2.192	4.0 Max	4.0 Max
Sulfur Sulfide (S), %	0.560	2.5 Max	2.5 Max
Chloride (Cl), %	pnd	----	----
(ASTM C 204) Blaine Fineness, m ² /km	524	----	----
(ASTM C 430) 325 Mesh, % Retained	99.8	20 Max	20 Max
Density	2.93	----	----
(ASTM C 185) Air Content, %	6.3	12 Max	12 Max
SLAG ACTIVITY INDEX, %			
7 Day Individual	134	70 Min	90 Min
7 Day Average of last 5	----	75 Min	95 Min
28 Day Individual	138	90 Min	110 Min
28 Day Average of last 5	----	95 Min	115 Min
REFERENCE CEMENT			
Total Alkali, %	0.721	0.60 - 0.90	0.60 - 0.90
(ASTM C 204) Blaine Fineness, m ² /km	382	----	----
C ₂ S, %	62.35	----	----
C ₃ S, %	12.28	----	----
C ₁ A, %	8.52	----	----
C ₄ A, %	7.2	----	----
COMPRESSIVE STRENGTH, psi			
7 Day Reference Cement	4890	----	----
28 Day Reference Cement	6260	5000 psi Min	5000 psi Min
7 Day Slag and Cement Reference	6560	----	----
28 Day Slag and Cement Reference	8660	----	----

This GGBFS meets the requirements of ASTM C 989-05 (Grade 100 and Grade 120)

Albert Miller

Heidelberg Technology Center - Research and Support



Holcim

Material Certification Report

Material: Portland Cement
Type: I-II(MH) (ASTM C 150)

Test Period: 01-Jun-2010
To: 30-Jun-2010

Certification

Holcim cement meets the specifications of ASTM C 150 for Type I-II(MH) cement.

General Information

Supplier: Holcim (US) Inc.
Address: 3500 State Highway 120
Florence, Co. 81226
Telephone: 719-784-1307
Date Issued: 12-Jul-2010

Source Location: Portland Plant
3500 State Highway 120
Florence, Co. 81226
Contact: Dick Roush

The following information is based on average test data during the test period. The data is typical of cement shipped by Holcim; individual shipments may vary.

Tests Data on ASTM Standard Requirements

Chemical			Physical		
Item	Limit ^A	Result	Item	Limit ^A	Result
SiO ₂ (%)	-	19.6	Air Content (%)	12 max	7
Al ₂ O ₃ (%)	6.0 max	4.7	Blaine Fineness (m ² /kg)	260 min 430 max	414
Fe ₂ O ₃ (%)	6.0 max	3.2	Autoclave Expansion (%) (C 151)	0.80 max	-0.02
CaO (%)	-	63.4	Compressive Strength MPa (psi)		
MgO (%)	6.0 max	1.5	3 days	10.0 (1450) min	31.1 (4510)
SO ₃ (%) ^C	3.0 max	3.4	7 days	17.0 (2470) min	37.6 (5460)
Loss on Ignition (%)	3.0 max	2.6	Initial Vicat (minutes)	45-375	126
Insoluble Residue (%)	0.75 max	0.59	Mortar Bar Expansion (%) (C 1038)		-0.001
CO ₂ (%)	-	1.4	Heat of Hydration 7 days, kJ/kg (cal/g) ^B		343 (82)
Limestone (%)	5.0 max	3.7			
CaCO ₃ in Limestone (%)	70 min	84			
Inorganic Processing Addition	5.0 max	0.0			
Potential Phase Compositions					
C ₃ S (%)	-	59			
C ₂ S (%)	-	11			
C ₃ A (%)	8 max	7			
C ₄ AF (%)	-	10			
C ₃ S + 4.75C ₄ A (%)	100 max	92			

Tests Data on ASTM Optional Requirements

Chemical			Physical		
Item	Limit ^A	Result	Item	Limit ^A	Result
Equivalent Alkalies (%)		0.70			

Notes

^A Dashes in the limit / result columns mean Not Applicable.

^B Test result represents most recent value and is provided for information only. Analysis of Heat of Hydration has been carried out by CTL Group, Skokie, IL.

^C It is permissible to exceed the specification limit provided ASTM C 1038 Mortar Bar Expansion does not exceed 0.02C %.

Adjusted per Annex A1.6 of ASTM C150 and AASHTO M85.

This data may have been reported on previous mill certificates. It is typical of the cement being currently shipped.

**Holcim****Material Certification Report**

Material: Portland Cement
Type: I-II(MH) (ASTM C 150)

Test Period: 01-Jun-2010
To: 30-Jun-2010

Certification

Holcim cement meets the specifications of ASTM C 150 for Type I-II(MH) cement

General Information

Supplier: Holcim (US) Inc
Address: 3500 State Highway 120
Florence, Co. 81226
Telephone: 719 784 1307
Date issued: 12 Jul 2010

Source Location: Portland Plant
3500 State Highway 120
Florence, Co. 81226
Contact: Dick Roush

The following information is based on average test data during the test period. The data is typical of cement shipped by Holcim; individual shipments may vary.

Item	Result ^A	Item	Result
Type	-	C ₁ S (%)	61
Amount (%)	-	C ₂ S (%)	11
SiO ₂ (%)	-	C ₃ A (%)	7
Al ₂ O ₃ (%)	-	C ₄ AF (%)	10
Fe ₂ O ₃ (%)	-		
CaO (%)	-		
SO ₃ (%)	-		

MIX 1 (#40)

POTENTIAL ALKALI REACTIVITY OF AGGREGATES, (MORTAR-BAR METHOD)
ASTM C 1567

RET BAR
940

Client: University of Colorado Denver
Project No. CT15238.000-400
Cement: Holcim Type I/II
Fly Ash: _____
Pozzolan: _____
Cement Alkalies: 0.70% (Total Alkalies as Na²O)
Autoclave Expansion: 0.00
Required Cement Content: 440 g
Aggregate/Cement Ratio: 990 g/440 g
Water/Cement Ratio: 0.47

Cast Date: April 4, 2011
Project: Alkali-Silica Reaction Testing
Coarse: Recycled Waste Glass
Fine #1: _____
Fine #2: _____
Lithium: _____
Soak Solution: _____

Water	206.8 g
Weight (g)	440
Cement: 100%	440
Fly Ash: _____	_____
Pozzolan: _____	_____
Waste Glass Coarse: 100%	_____
Fine #1: _____	_____
Fine #2: _____	_____

Passing	Retained	Mass (%)	Weight (g)	Coarse: Weight (g)	Fine #1: Weight (g)	Fine #2: Weight (g)
No. 4 (4.75 mm)	No. 8 (2.36 mm)	10%	99.0			
No. 8 (2.36 mm)	No. 16 (1.18 mm)	25%	247.5			
No. 16 (1.18 mm)	No. 30 (600 µm)	25%	247.5			
No. 30 (600 µm)	No. 50 (300 µm)	25%	247.5			
No. 50 (300 µm)	No. 100 (150 µm)	15%	148.5			

Longitudinal Measurement Data

	Date	Sample I.D.			Average
		1	2	3	
Initial Reading (24 hr)	4/5/11	1009	0960	1181	
Zero Reading (48 hr)	4/6/11	1067	1021	1240	
3 Day Reading	4/9/11	1067	1023	1240	
Percent Change	-	0.000	0.002	0.000	0.001
7 Day Reading	4/13/11	1115	1081	1266	
Percent Change	-	0.048	0.060	0.026	0.045
10 Day Reading	4/16/11	1247	1215	1301	
Percent Change	-	0.180	0.192	0.061	0.144
14 Day Reading	4/20/11	1305	1420	1479	
Percent Change	-	0.238	0.399	0.239	0.292
21 Day Reading	4/27/11				
Percent Change	-				
28 Day Reading	5/4/11				
Percent Change	-				

DRAFT

88

Mix #34

POTENTIAL ALKALI REACTIVITY OF AGGREGATES, (MORTAR-BAR METHOD)
ASTM C 1567

Client: University of Colorado Denver
Project No. CT15238.000-400
Cement: Holcim Type I/II
Fly Ash: Lehigh All Chem Grade 120
Pozzolan: ~~Lehigh~~
Cement Alkalies: 0.70% (Total Alkalies as Na₂O)
Autoclave Expansion: 0.00
Required Cement Content: 440 g
Aggregate/Cement Ratio: 990 g/440 g
Water/Cement Ratio: 0.47

Cast Date: April 4, 2011
Project: Alkali-Silica Reaction Testing
Coarse: Recycled Waste Glass
Fine #1:
Fine #2:
Lithium:
Soak Solution:

Water 206.8 g

	Weight (g)
Cement:	50%
Fly Ash:	50%
Pozzolan:	

Waste Glass Coarse: 100%
Fine #1:
Fine #2:

Passing	Retained	Mass (%)	Coarse: Weight (g)	Fine #1: Weight (g)	Fine #2: Weight (g)
No. 4 (4.75 mm)	No. 8 (2.36 mm)	10%	99.0		
No. 8 (2.36 mm)	No. 16 (1.18 mm)	25%	247.5		
No. 16 (1.18 mm)	No. 30 (600 µm)	25%	247.5		
No. 30 (600 µm)	No. 50 (300 µm)	25%	247.5		
No. 50 (300 µm)	No. 100 (150 µm)	15%	148.5		

Longitudinal Measurement Data

	Date	Sample I.D.			Average
		1	2	3	
Initial Reading (24 hr)	4/5/11	1485	1110	0943	
Zero Reading (48 hr)	4/6/11	1549	1102	1002	
3 Day Reading	4/9/11	1543	1165	1002	
Percent Change	-	0.000	0.003	0.000	0.001
7 Day Reading	4/13/11	1560	1183	1018	
Percent Change	-	0.017	0.021	0.016	0.018
10 Day Reading	4/16/11	1561	1185	1020	
Percent Change	-	0.018	0.023	0.020	0.020
14 Day Reading	4/20/11	1561	1190	1020	
Percent Change	-	0.018	0.028	0.020	0.022
21 Day Reading	4/27/11				
Percent Change	-				
28 Day Reading	5/4/11				
Percent Change	-				

DRAFT

* SEE BELOW
MIX 3 #33

POTENTIAL ALKALI REACTIVITY OF AGGREGATES, (MORTAR-BAR METHOD)
ASTM C 1567

PUT DATA SHEET IN NEXT
DAY SLOT & SET TIME VARIANCE

REF BAR 940
Client: University of Colorado Denver
Project No. CT15238.000-400
Cement:
Fly Ash: ~~Lehigh~~ All Chem Grade 120
Pozzolan:
Cement Alkalies: 0.70% (Total Alkalies as Na₂O)
Autoclave Expansion: 0.00
Required Cement Content: 440 g
Aggregate/Cement Ratio: 990 g/440 g
Water/Cement Ratio: 0.47
Cast Date: April 4, 2011
Project: Alkali-Silica Reaction Testing
Coarse: Recycled Waste Glass
Fine #1:
Fine #2:
Lithium:
Soak Solution:
Water: 206.8 g

GG BFS
Cement:
Fly Ash: 100%
Pozzolan:
Waste Glass Coarse: 100%
Fine #1:
Fine #2:

Passing	Retained	Mass (%)	Coarse: Weight (g)	Fine #1: Weight (g)	Fine #2: Weight (g)
No. 4 (4.75 mm)	No. 8 (2.36 mm)	10%	99.0		
No. 8 (2.36 mm)	No. 16 (1.18 mm)	25%	247.5		
No. 16 (1.18 mm)	No. 30 (600 µm)	25%	247.5		
No. 30 (600 µm)	No. 50 (300 µm)	25%	247.5		
No. 50 (300 µm)	No. 100 (150 µm)	15%	148.5		

Longitudinal Measurement Data

	Date	Sample I.D.			Average
		1	2	3	
Initial Reading (24 hr)	4/6/11	1135	1144	1278	
Zero Reading (48 hr)	4/6/11	1197	1204	1338	
3 Day Reading	4/9/11	1199	1204	1338	
Percent Change	-4	0.002	0.000	0.000	0.001
7 Day Reading	4/13/11	1215	1222	1368	
Percent Change	-	0.018	0.018	0.030	0.022
10 Day Reading	4/16/11	1215	1222	1368	
Percent Change	-	0.018	0.018	0.030	0.022
14 Day Reading	4/20/11	1215	1222	1368	
Percent Change	-	0.018	0.018	0.030	0.022
21 Day Reading	4/27/11				
Percent Change	-				
28 Day Reading	5/4/11				
Percent Change	-				

* MIX THREE HAD AN EXTRA CURE DAY
DUE TO EXTENDED SET
TIME MIX. CLIENT AND D. BARNETT
SHADE

DRAFT